**US Army Corps
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Demonstration Erosion Control
Project Monitoring Program

Fiscal Year 1993 Report

**Volume VI: Appendix E
Model Study of the Demonstration
Erosion Control 10-ft Riprap Drop
Grade Control Structure**

by   Sandra K. Martin, Sheila F. Knight,
Thomas E. Murphy

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Final report

**US Army Corps
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STRUCTURES
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SCALE
900   0   900 m

AREA OF RESERVATION = 2.7 sq km

Contents

1 Introduction

Background

Low-drop grade control structures have been used to arrest erosion in incising channels. The concept of the drop structure was originally developed based on an equivalent energy approach. Numerous variations and types of these structures have been constructed both in model studies and in prototype locations.

Sheet-pile grade control structures have been used in the Demonstration Erosion Control (DEC) Project in the Yazoo Basin, Mississippi, to arrest erosion due to headcutting. These structures consist of an upstream approach transition section from the natural channel to the sheet-pile weir, a vertical drop into a riprap stilling basin to dissipate the energy, and a downstream transition section that ties back into the natural channel. The sheet-pile and riprap approach to low-drop design is an economical alternative to a concrete structure and apron.

Purpose and Approach

Current design criteria[1] for a sheet-pile grade control structure limits the drop height to 6 ft. The limits are partially based on hydraulic limitations and partially on structural design limitations of the vertical placement of the sheet-pile cutoff. Due to the potential for cost savings with a sheet-pile structure as opposed to a concrete drop structure, a re-evaluation of structural design components by the Vicksburg District verified the constructability of a higher drop (10 ft). However, the hydraulic performance and riprap design criteria were not heretofore tested for the Agricultural Research Service's (ARS) low-drop structure nor design criteria developed for sheet-pile riprap drops greater than 6 ft.

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[1] Little, W. C., and Murphy, J. B. (1982). "Model study of low drop grade control structures," *Journal of the Hydraulics Division*, ASCE, 108(HY10), 1132-1146.

Drop structures have typically been classified either as low or high drops according to a ratio of drop height, H, to critical depth, Y_c. Low drops are classified as those with a H/Y_c less than or equal to 1. The proposed drop height of 10 ft would change the classification of drop structure, for the same design discharge and critical depth of 6 ft, by exceeding 1. Therefore, based on the differences between the actual drop classification and the proposed design criteria, it was necessary to study the hydraulic performance of this structure.

The purpose of this study was to modify and/or develop guidance regarding both the hydraulic design and the stable riprap design to accommodate a 10-ft drop structure with an H/Y_c greater than 1. The objective of the study was to determine the feasibility of using a higher drop and develop design guidance pertaining to the higher drop. A 1:12-scale physical model was used to investigate the proposed sheet-pile grade control structure with a 10-ft drop.

2 Design Assumptions

The drop structure design was based on the modified ARS-type structure previously recommended in a study conducted by Colorado State University (CSU). The dimensions were determined from the ARS criteria and the CSU study, along with recommendations by the Vicksburg District. The original basin design dimensions and criteria were selected such that results from the CSU model and the US Army Engineer Waterways Experiment Station (WES) model would be comparable.[1]

Many of the design dimensions were contingent upon the critical depth; therefore, a design discharge of 4,000 cfs was selected. This same design discharge had been used in the previous model by CSU. A channel bottom width and weir length of 40 ft was selected. The weir shape was trapezoidal with 2.5:1 side slopes. The critical depth based on the weir cross-sectional shape and the discharge was 6.0 ft. All design dimensions that are a function of critical depth were based on 6.0 ft. The channel drop, H, for design was 10 ft.

The basin design criteria deviates slightly from that developed by Little and Murphy[2] according to actual prototype structures used in the DEC. Specifically, a trapezoidal stilling basin replaced the wider and more rounded planform; the drop was vertical instead of sloping; the baffle plate was not used; and the location of the larger riprap was based on the critical areas identified in the CSU study.

Drop Structure Dimensions

The dimensions were determined from the following criteria (notation

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[1] Abt, Steven R., Watson, Chester C., Johns, Derek D., Hamilton, Glenn B., Garton, Andrew D., Florentin, C. Bradley, and Thornton, Christopher I. (1991). "Riprap sizing criteria for ARS-type drop structures," prepared by the Department of Civil Engineering, Colorado State University, Fort Collins, CO, for U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

[2] Op. Cit.

adapted from Abt et al.[1]). The weir was set at a fictitious prototype elevation of 100 ft. The drop plan and profile dimensions are shown in Figures 1 and 2.

Given:

The design discharge, Q

$$Q = 4000 \ cfs \tag{1}$$

The channel width and weir length, B

$$B = 40 \ ft \tag{2}$$

The stilling basins side slopes, S_B

$$S_B = 2.5H:1V \tag{3}$$

The end sill slope, S_E

$$S_E = 5H:1V \tag{4}$$

Calculate:

The variable, X_B

$$X_B = Y_c \left[3.54 + 4.26 \left(\frac{H}{Y_c} \right) \right] \tag{5}$$

The stilling basin length, L_{SB}

$$L_{SB} = 2X_B \tag{6}$$

The stilling basin depth, Y_{SB}

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[1] Op. cit.

$$Y_{so} = Y_c + H \tag{7}$$

Riprap

The previous study by CSU recommended that two gradations of riprap be used in the drop structure design. The larger gradation is placed immediately downstream of the weir and along the basin floor, while the smaller is placed on the remaining side slopes and in the approach. The specific dimensions and placement can be seen in Figures 1 and 2.

Based on guidance from the Vicksburg District, two gradations of riprap were originally selected. The gradation envelopes for the riprap used in the model were obtained from a Lower Mississippi Valley Division document, "Report on the Standardization of Riprap."[1] The thickness, based on highly turbulent flow, for the stone having an upper limit of 1,500 lb (R1500) and the stone having an upper limit of 200 lb (R200) was 48 in. and 24 in., respectively. These gradations are common to the Vicksburg District area. The gradations are as follows for a specific weight of 155 pcf:

Percent Lighter by Weight	R1500 Stone Size		R200 Stone Size	
	Upper	Lower	Upper	Lower
100	1500	600	200	80
50	650	300	80	40
15	330	100	40	10

At the request of the Vicksburg District, a larger gradation was used in place of the R1500 stone following the first set of tests. This stone size having an upper limit of 2,200 lb (R2200), is placed at a minimum thickness of 54 in. The gradation for a specific weight of 155 pcf is:

Percent Lighter by Weight	R2200 Stone Size	
	Upper	Lower
100	2200	900
50	930	440
15	460	130

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[1] U.S. Army Engineer Division, Lower Mississippi Valley. (1981 (revised Jan 1982)). "Report on standardization of riprap gradations," Vicksburg, MS.

Flow Conditions

Discharges studied ranged from 2,000 cfs to 8,000 cfs with a design discharge of 4,000 cfs. When testing began for the DEC study, the maximum prototype discharge that could be simulated in the physical model was 5,300 cfs. Modifications were later made to carry a higher discharge. Tailwater conditions were varied from a low elevation of 95.4 ft (4.6 ft below weir crest) at 2,000 cfs to a maximum elevation of 109 ft (9 ft above weir crest).

3 The Model

Description

A 1:12-scale model of a 10-ft drop grade control structure proposed for the DEC project was constructed to test riprap stability for various flow conditions. The model, shown in Figures 3-5, reproduces approximately 400 ft of the prototype approach channel, the weir, stilling basin and end sill, and approximately 320 ft of downstream channel. The dimensions of the drop control structure were based on a design discharge of 4,000 cfs and criteria previously tested by CSU.[1] The upstream and downstream channels were constructed by molding sand and cement mortar to sheet metal templates. The weir was constructed from plywood. The stilling basin was constructed with sand, and graded rock was placed over a filter cloth. A layout of the model is shown in Figure 6 in prototype dimensions.

Model Appurtenances

The water was supplied by a circulating system and discharges were measured with a venturi meter. Velocities were measured with a one-dimensional propeller type electronic velocity meter. Water surfaces were recorded using piezometers. The locations of piezometers used for water-surface measurements and locations where the velocities were recorded are shown in Figure 7. Tailwater conditions were regulated by adjusting a tailgate until the most downstream piezometer was reading the desired tailwater elevation. Flow conditions were recorded photographically and with video camera. Velocities in the upstream and downstream channel sections were measured for each condition. Photos were also obtained when riprap displacement occurred in the stilling basin exposing the filter cloth.

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[1] Abt et al., op. cit.

Scale Relations

The equations of hydraulic similitude, based on the Froude criteria, were used to express mathematical relations between the dimensions of hydraulic quantities of the model and prototype. General relations for transferring model data to prototype equivalents are as follows:

Characteristic	Dimension*	Scale Relations Model:Prototype
Length	L_r	1:12
Area	$A_r = L_r^2$	1:144
Velocity	$V_r = L_r^{1/2}$	1:3.464
Time	$T_r = L_r^{1/2}$	1:3.464
Discharge	$Q_r = L_r^{5/2}$	1:498.83
Weight	$W_r = L_r^3$	1:1,728
Roughness Coefficient	$N_r = L_r^{1/6}$	1:1.513

Using a model stone with a specific weight of 167 pcf, stone sizes in the model were conservatively selected toward the lower side of the gradation envelope. The equivalent spherical diameter at 50% passing by weight, d_{50}, of each gradation tested in the model at prototype and model dimensions is as follows:

Stone Size Type	Prototype d_{50}, ft	Model d_{50}, in
R200	0.75	0.75
R1500	1.50	1.50
R2200	2.00	2.00

4 Tests and Results

The same stilling basin design, Type 1, was used throughout testing. Two different weir designs were used; a trapezoidal weir, Type 1 Design Weir and a rectangular weir, Type 2 Design Weir. The small stone (R200) remained constant in the described locations throughout testing. In the areas containing large stone, however, two gradations of stone, R1500 and R2200, were tested. In one series of tests, the R1500 gradation was grouted.

The small section of riprap placed immediately upstream of the weir was grouted due to riprap failure occurring for low tailwater conditions. During testing it was found that velocities in that area exceeded 16 fps with a discharge of 4,000.

Riprap failure was defined as the condition where sufficient stone displacement occurred to expose the underlying filter cloth.

A total of 93 tests were conducted for this study. Table 1, summary of test conditions, contains summary information regarding each test.

Velocity and profile data for certain tests are provided during the discussion. These data are provided to evaluate the flow conditions in the approach and exit channels.

Type 1 Design Stilling Basin and Type 1 Design Weir

Initial tests (Tests 1-13) were conducted with the Type 1 design basin (Figure 1) and Type 1 design weir (Figure 8). All tests were run for a time period of 7 hours prototype. With a discharge of 2,000 cfs and tailwater elevation of 109.5, surface jet type flow conditions existed, as shown in Figure 9. The downstream water surface was relatively smooth and the riprap remained stable. Water-surface elevations and velocity measurements for this condition are shown in Figure 10. The discharge was increased to 2,500 cfs with the tailwater elevation remaining at 109.5. Data similar to those obtained with a discharge of 2,000 cfs are shown in Figure 11. The riprap again remained stable.

When the discharge was increased slightly to 3000 cfs, changes were observed in the downstream water surface. Surface flow dominated the tailwater conditions and small vortices shedding off both sides of the weir were present in the stilling basin (Figure 12). These vortices did not cause significant turbulence directly over the riprap due to the depth of tailwater, and the riprap remained stable. Small eddies also formed over the side slopes of the stilling basin downstream of the weir. A water-surface profile and velocity measurements are provided in Figure 13.

The discharge was increased to 3,500 cfs with the tailwater elevation remaining at 109.5. Velocities increased, but flow conditions were similar to those observed with 3,000 cfs.

Tests were then conducted with the design discharge of 4,000 cfs. With the tailwater at el 109.5, flow conditions were similar to those observed with a discharge of 3,500 cfs with the same tailwater conditions, and the riprap remained stable (Figures 14 and 15).

The next series of tests consisted of lowering the tailwater elevation with a discharge of 4,000 cfs until riprap failure occurred. When the tailwater was lowered to el 108.0, increases in water-surface roughness and velocities were observed, but the riprap remained stable. As the tailwater was lowered to el 105.0, velocities and water-surface roughness over the stilling basin increased, as did the vortex activity and turbulence from the corners of the weir, as would be expected. However, the riprap still remained stable. Velocities and water-surface profiles for these conditions are provided in Figures 16-19. Figure 20 is a photo of flow conditions with a tailwater elevation of 105.0.

When the tailwater was lowered to el 104.0 (Figures 21 and 22), an increase in the size of the surface waves, tighter eddies along each side of the basin, and displacement of some stones from the stilling basin to the downstream exit channel were observed, but no filter cloth was exposed. At a tailwater elevation of 103.0 (Figures 23 and 24), an undulating type hydraulic jump formed in the stilling basin which contributed to an increase in downstream surface waves. It was also observed that similar stone displacement occurred at this tailwater elevation as compared to tests conducted with the tailwater elevation at 104.0. With a tailwater elevation of 102.0, a very weak hydraulic jump formed in the stilling basin and there was some decrease in downstream surface waves, as shown in Figure 25. Stone displacement was similar to observations noted at tailwater el 103.0. Velocity measurements and a water-surface profile are shown in Figure 26.

Failure in the R1500 riprap gradation occurred when the tailwater elevation was lowered to 101.0 with a discharge of 4,000 cfs. The hydraulic jump strengthened and tight side eddies formed downstream of the sides of the weir (Figure 27). The locations where the riprap failure occurred are indicated in Figure 28. Figure 29 is a photograph showing the exposed filter cloth for the above condition, and Figure 30 shows velocities and a water-surface profile for this flow condition.

Tests results with the design discharge of 4,000 cfs, the Type 1 design basin and R1500 stone indicated that the stilling basin riprap gradations remained stable with no movement or displacement with tailwater elevations ranging down to 105.0. Using a value for critical depth of 6.0 ft for the Type 1 design weir and the submergence criteria established for the CSU study, the submergence value is 0.83. Actual riprap failure occurred at a tailwater elevation of 101.0 with the Type 1 design weir.

Testing was continued with the Type 1 design weir and the larger riprap gradation. The R2200 riprap was used in place of all R1500 stone. The same structural shape and dimensions were used for these tests as was the placement of the R200 stone.

Discharges from 2,000 to 8,000 cfs were tested in 500 cfs increments (tests 40-103). At each discharge, the tailwater was lowered at 1-ft increments to define the point where riprap failure occurred. Both the large riprap (R2200) and the small riprap (R200) were observed for these conditions.

In previous tests where the discharge was only taken up to 5,300 cfs, the small rock was not observed to experience failure. However, at high discharges and at certain tailwater conditions, the small rock exhibited failure. In some cases the small rock failed at a higher tailwater than the large rock. Failure conditions of both the large and small stone were documented and considered in the analysis and final conclusions.

When failure occurred on either gradation, the location essentially remained in the same area of the stilling basin. The dimensions of the failure areas and sometimes the side slope on which they were observed varied for different test conditions.

The R2200 stone did not fail at the design discharge of 4,000 and a tailwater elevation as low as 97.7. No movement was observed at a tailwater of 102.0. The submergence at this elevation was 0.33.

Type 1 Design Stilling Basin and Type 2 Design Weir

The shape of the weir was modified in an effort to improve flow conditions in the stilling basin. The weir was changed from a trapezoidal to a rectangular shape, as shown in Figure 8. As a result of the restricted weir section, the water-surface elevation increased in the approach channel. Again, testing was resumed at a discharge of 4,000 cfs and the tailwater was lowered until failure of the riprap occurred. With the tailwater elevation set at 109.0, eddies formed on each side of the stilling basin downstream of the weir and flow was directed more toward the center of the channel than with the Type 1 design weir, as shown in Figure 31. The flow downstream of the weir shifted from one side of the basin to the other before stabilizing in the exit channel. This

type of oscillating flow pattern occurred for each tailwater elevation observed with the Type 2 design weir in place. The riprap remained stable for a tailwater elevation of 109.0. Velocities and a water-surface profile are shown in Figure 32.

The tailwater was then lowered to el 108.0. Surface flow conditions were similar to those observed at tailwater el 109.0; however, velocity magnitudes and turbulence increased sufficiently to displace some stone from the invert of the stilling basin to the downstream exit channel. Water-surface and velocity measurements (Figure 33) indicated that the previously observed surface jet flow was beginning to plunge and a submerged jet was beginning to form over the riprap basin.

With a tailwater elevation of 107.0, the velocities again increased near the invert of the stilling basin over the riprap (Figure 34), and some stones were displaced to the exit channel. Surface flow conditions were similar to those observed at tailwater el 108.0, except for an increase in downstream surface wave action.

The tailwater elevation was lowered to 105.0 forming surface waves with an increase in velocities along the bottom over the downstream end of the basin exit channel, as shown in Figures 35 and 36. A number of stones were displaced from the basin floor 25 to 40 ft downstream of the weir, indicating the jet flow over the weir was plunging through the tailwater. Some stones were also displaced near the end of the stilling basin; however, no cloth was exposed. Flow conditions and stone displacement were similar at a tailwater elevation of 104.0. A velocity of 10.9 fps was measured near the bottom at the end of the stilling basin, again indicating the presence of a submerged jet (Figure 37).

When the tailwater was lowered to el 103.0, a large section of R1500 riprap failed (Figures 38 and 39). Surface flow conditions were similar to those observed at tailwater el 104.0, with a slightly more plunging jet type flow just downstream of the weir (Figure 40). Velocities at the end of the stilling basin were more evenly distributed throughout the depth of flow, as shown in Figure 41. This was due to the energy dissipation that occurred over the scour area.

The rectangular weir (Type 2 design) moved the failure zone off the side slopes. The energy increased in the stilling basin due to the restricted cross-sectional area of the rectangular weir resulting in riprap failure on the basin floor at a higher submergence of 0.44 (trapezoidal weir, 0.17).

Velocities over the type 2 design weir were lower than velocities over the Type 1 (trapezoidal) design weir at a tailwater elevation of 108 (compare Figures 16 and 33). As the submergence was lowered, velocities over the Type 2 design weir were comparable to and sometimes higher than those measured over the Type 1 design weir, as can be seen by comparing Figures 17, 18, 19, 23, 26, and 30 with Figures 34, 36, 37, and 41. The

water-surface elevation in the approach channel was higher with the rectangular weir in place; therefore, velocities were lower in the approach channel upstream of the weir.

For the design discharge of 4,000 cfs, the R1500 stone remained stable with no movement with tailwater elevations ranging down to 108.0 with the Type 2 design weir. Using the CSU submergence criteria and a critical depth value for the rectangular weir of 6.8 ft, the submergence for this conditions is 1.18. Failure occurred at a tailwater elevation of 103.0.

Type 1 Design Weir and Type 1 Design Stilling Basin, Grouted Section of Riprap

Following the riprap failure observed in evaluation of the Type 2 design weir, further testing was conducted with the Type 1 design weir. Tests were conducted to determine the effectiveness of grouting a portion of the riprap below the Type 1 design weir, as shown in Figure 42. Initial tests were conducted with a discharge of 4,000 cfs. Velocity profiles and water-surface elevations for a discharge of 4,000 cfs with tailwater elevations of 101.0 and above were not measured with the grout in place. Conditions will be the same as Type 1 weir, no grout.

The tailwater elevation was set at 103.0, and as in previous tests, an undular-type jump formed in the basin with surface waves in the downstream channel. A small number of the small size (R200) stones were displaced from the side slopes immediately downstream of the grouted area. The stone was displaced upstream to the grouted area. This minor displacement may have been taking place in previous tests without the grout, but went unnoticed since it blended in with the existing stone in the area.

The tailwater was then lowered to el 102.0 and a weak hydraulic jump formed in the basin with a reduction in downstream wave action. Strong eddies formed on each side of the jump and approximately 20 to 30 of the R200 gradation stones from the side slopes were displaced upstream to the grouted area

With a tailwater elevation of 101.0, a strong hydraulic jump formed over the grouted area and downstream surface waves were reduced. Strong, tight eddies formed on each side of the jump and a significantly larger number of the R200 gradation stones were displaced from the side slopes immediately downstream of the grout upstream to the grouted area, but no cloth was exposed.

When the tailwater was lowered to el 100.0, the jet plunged over the weir forming a strong hydraulic jump with good energy dissipation over the grouted area. The improved hydraulic conditions resulted in fewer stones being displaced from the side slopes downstream of the grout than were observed at

the higher submergence previously tested. The water surface in the exit channel was relatively smooth and velocities in this area were fairly uniform throughout the depth of flow. Similar flow conditions were observed with a tailwater elevation of 99.0. Velocity and water-surface measurements for tailwater elevations of 100.0 and 99.0 are shown in Figures 43 and 44, respectively. A tailwater elevation of 99.0 was the lowest that could be obtained in the model for a discharge of 4,000 cfs.

Additional tests were conducted at a discharge of 5,300 cfs to determine the riprap stability under these extreme conditions. Tests were initially conducted with a tailwater elevation of 109.0. The flow jet did not plunge at this tailwater elevation, but remained along the surface creating surface waves over the stilling basin area (Figure 45). The riprap did remain stable for these tests. Velocities and a water-surface profile for the above flow condition are shown in Figure 46.

When the tailwater was lowered to el 108.0 and 107.0 (Figures 47 and 48), the downstream surface waves increased and the thickness of the jet over the weir noticeably increased compared to flow conditions observed with a discharge of 4,000 cfs. This resulted in increased turbulence over the basin area. Some small to medium size R200 gradation stones were displaced upstream to the grouted area. Some of the larger R1500 gradation stones were displaced from the end sill to the downstream exit channel. However, no filter cloth was exposed. Figure 49 is a photograph of flow conditions with tailwater el 107.0.

The tailwater elevation was lowered to 106.0. An undular jump formed creating large surface waves downstream of the weir. Stone displacement was from the same areas as observed with higher tailwater elevations, but the number of stones displaced increased. In some areas of the side slopes just downstream of the grouted riprap, the R200 gradation was reduced to a one-stone thickness over the filter cloth, further indicating poor hydraulic conditions present in the stilling basin. Velocities and a water-surface profile for the above condition are shown in Figure 50.

Failure of the riprap occurred when the tailwater was lowered to el 105.0. An undular jump formed with strong eddies on both sides of the stilling basin and large surface waves present downstream, as shown in Figure 51. The plunging flow from the sides of the weir created sufficient turbulence on the side slopes downstream of the grouted section to fail patches of the R200 riprap on the left and right side slopes. The movement of stone was mainly in the downstream direction, but some stones were displaced from the end sill area to the exit channel. Locations where failure occurred are provided in Figures 52 and 53. Velocities and a water-surface profile for a tailwater elevation of 105.0 are shown in Figure 54.

A section of grouted riprap below the Type 1 design weir allowed for lower submergences without failure of the ungrouted riprap for a discharge of 4,000 cfs. When a strong hydraulic jump formed over the grouted section of

riprap, good energy dissipation was observed in this area and no riprap failure occurred. Although no riprap failure occurred with a discharge of 4,000 cfs, moderate riprap displacement was recorded at tailwater elevations between 103.0 and 99.0. This instability would indicate the need to extend the grouted riprap further downstream.

The critical depth for a discharge of 5,300 cfs is 6.9 ft. With the grouted riprap in place, the loose riprap was unstable in the stilling basin for tailwater elevations below 109.9 and failed at a tailwater elevation of 105.0. The submergence at these tailwaters are 1.30 and 0.27, respectively.

5 Analysis

Analyses of the data were performed according to the same methodologies and criteria used by CSU in their previous evaluation of a 6-ft drop grade control structure. Evaluation of the test results in this manner allowed for the comparison of recommended design equations. The tests used for the analysis of the riprap included tests 1-13 and tests 40-103. In the CSU study, a riprap design equation was developed to determine the minimum stone requirements for a vertical drop structure with a trapezoidal weir.

The final recommended design method by CSU related the D50 to unit discharge (q), critical depth (Yc) and submergence (S). Unit discharge was defined in this study as the design discharge divided by the length of weir. For a rectangular weir this accurately reflects the discharge per unit foot of flow. For a trapezoidal weir, Type 1 weir design, the unit q determined by dividing the total discharge by the bottom width of the weir, actually results in a higher unit representation of the flow conditions because it essentially ignores the flow area over the slope. However, this method was agreed upon between the Vicksburg District and CSU for the 6-ft drop and was therefore adopted by WES in the analysis of the 10-ft drop. Using this definition precluded use of the results for loose riprap from the type 2 weir design tests. (Tests using grout were not included either.)

The theoretical values of critical depth and normal depth (Yn) were calculated using the methodology described in Chow's *Open-Channel Hydraulics*[1] and are found in Tables 2 and 3 of test results. The calculations for normal depth assumed a stream slope of 0.0024, side slopes of 1:1, and a Manning's n value of 0.035. Critical depth calculations were used in the analysis to determine submergence.

Submergence is defined as the elevation of the tailwater minus the elevation of the weir all over critical depth. This can be a negative value if the tailwater elevation is below the weir.

Values for q/D50 and submergence from Tables 2 and 3 (large riprap stability and small riprap stability) were plotted to establish a relationship

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[1] Chow, Ven Te. (1959). *Open-Channel Hydraulics*, McGraw-Hill, New York.

between these variables for both the large riprap and the small riprap. The stability of the large stone was evaluated first. Figure 55 shows the data points for the large riprap. An exponential curve was fitted through the data to define a threshold between the stable and not stable riprap. The exponential form of the equation was based on the CSU study. The coefficients were determined using a statistical software package to fit the observations at the threshold to the model. The final design equation for the 10 ft drop is:

$$\frac{q}{d_{50}} = 38.53 \, e^{0.91 \, s} \tag{8}$$

The equation recommended by CSU for the 6 ft drop was:

$$\frac{q}{d_{50}} = 31.97 \, e^{1.55 \, s} \tag{9}$$

Comparing the two design equations, Figure 56, it can be seen that there appears to be some discrepancy on the lower end of the curve. That is, for values of submergence of approximately 0.3 or less, the 10-ft drop curve gives a smaller, or less conservative, stone size for a given unit discharge and submergence. Upon evaluation of the test data used by CSU, it was determined that testing was not conducted at as many lower submergence values as it was for this study. Therefore, more data points were used to define the curve, giving it more creditability at the lower submergence.

The design guidance recommended by CSU also included a safety factor. That is, they determined the shift in the design D50 due to outliers from their testing and concluded that the safety factor should be 1.25. Values of D50 determined by the equation should be multiplied by 1.25. Similar analysis of outlying points from the testing on the 10-ft drop came to the same recommended safety factor of 1.25.

In the analysis of the small stone it was determined that at higher discharges, coupled with the higher drop, a large circular eddy tended to fail the small stone at the water line. This failure resulted in an eventual slough of materials down slope and a mass failure of the small rock.

Therefore, special guidelines should be observed in the design of the small stone. First, the absolute minimum rock size should be the R200 gradation. Second, design conditions for this rock design should fall within the observed stable conditions. Testing concluded that the maximum q for submergences of approximately 0.33 or less is 100 cfs/ft for a stable condition. This discharge represents the design q for this drop structure.

Figure 57 shows the curve which bounds the stable and unstable conditions for the small riprap. Testing was not conducted which varied the size of the small stone; therefore, stone sizes are indeterminable for values of q in excess of 100 at submergences of 0.33 or less.

6 Conclusions and Recommendations

Hydraulic Conditions

In general, velocities in the approach channel increased with increasing discharges. Velocities in the exit channel increased with decreasing submergence. To insure stability of the approach and exit channels more consideration should be given to the velocities in these areas.

Tests performed using the Type 2 weir (rectangular) seemed to indicate that energy dissipation was more confined to the stilling basin than during the trapezoidal weir tests. While this leads to an earlier failure of the stone below the weir, velocities were lower in the exit channel. Furthermore, upstream of the weir, the water was pooled behind the weir causing eddies to form on each side of the approach channel. Since pool levels were higher with this design, approach velocities were lower. Optimum hydraulic performance would dictate that the transition from the natural channel into the weir should be a controlled contraction of the channel to the weir. However, due to the limitations regarding stone placement on a 2.5:1 slope in the upstream approach this may not be feasible.

Riprap Stability

In areas where loose stone was grouted in the model, both upstream of the weir and in the stilling basin, no failure of the stone occurred. Since there is a risk of mass failure due to uplift, consideration of this option should be based on field success.

Due to the instability of the small stone at the higher discharge during the grouted basin tests, consideration should be given to either extending the grouted section of rock or increasing the small stone size. These tests also verified the need to study the effects of riprap stability with discharges greater than the design.

Beyond the guidelines described in the analysis section, a design equation is not recommended for the small riprap until further studies are conducted. Furthermore, tests should be conducted to verify the effects of riprap stability with discharges greater than design.

Design of the large stone for the 10-ft drop should be based on the modified equation for large stone developed from this testing. (See Equation 8.)

A properly designed granular filter of some type should be provided beneath the graded riprap to prevent piping through voids in the rock.

Further studies should consider the following:

a. Optimize the dimensions of the approach to minimize upstream velocities.

b. Consider combinations of loose stone, gabions, and concrete protection in design of the structures.

c. Consider modifications to the stilling basin dimensions.

d. Lengthening the stilling basin and adding energy dissipating baffle "blocks".

e. Take into account the discrepancies in defining unit q for trapezoidal and rectangular weirs.

f. Model environmentally enhancing alternatives such as rock dikes in the stilling basin.

g. Take detailed velocity measurements in the stilling basin to facilitate calibration of numerical models.

h. Conduct more tests to determine the stable design criteria for the small riprap, especially at discharges in excess of the design discharge.

In summary, the objective of this study were accomplished by providing an equation for design of stone in a 10-ft drop structure and guidance on hydraulic design of a rectangular shaped weir. However, some caution should be exhibited in the implementation of the structure in a movable bed channel, especially with regard to the upstream approach and the downstream exit channel.

Table E1
Summary of Test Conditions

Test #	Basin Conditions	Weir Type	Discharge	Tailwater
1	Loose Stone, R1500	1	2000	109.5
2	Loose Stone, R1500	1	2500	109.5
3	Loose Stone, R1500	1	3000	109.5
4	Loose Stone, R1500	1	3500	109.5
5	Loose Stone, R1500	1	4000	109.5
6	Loose Stone, R1500	1	4000	108.0
7	Loose Stone, R1500	1	4000	107.0
8	Loose Stone, R1500	1	4000	106.0
9	Loose Stone, R1500	1	4000	105.0
10	Loose Stone, R1500	1	4000	104.0
11	Loose Stone, R1500	1	4000	103.0
12	Loose Stone, R1500	1	4000	102.0
13	Loose Stone, R1500	1	4000	101.0
14	Loose Stone, R1500	2	4000	105.0
15	Loose Stone, R1500	2	4000	107.0
16	Loose Stone, R1500	2	4000	109.0
17	Loose Stone, R1500	2	4000	108.0
18	Loose Stone, R1500	2	4000	104.0
19	Loose Stone, R1500	2	4000	103.0
20	Grouted Stone, R1500	1	4000	102.0
21	Grouted Stone, R1500	1	4000	103.0
22	Grouted Stone, R1500	1	4000	101.0
23	Grouted Stone, R1500	1	4000	100.0
24	Grouted Stone, R1500	1	4000	99.0
25	Grouted Stone, R1500	1	5300	105.0
26	Grouted Stone, R1500	1	5300	108.0
27	Grouted Stone, R1500	1	5300	109.0
28	Grouted Stone, R1500	1	5300	107.0
29	Grouted Stone, R1500	1	5300	106.0
40	Loose Stone, R2200	1	3500	105.0
41	Loose Stone, R2200	1	3500	104.0
42	Loose Stone, R2200	1	3500	103.0

(Sheet 1 of 3)

Table E1 (Continued)				
Test #	Basin Conditions	Weir Type	Discharge	Tailwater
43	Loose Stone, R2200	1	3500	102.0
44	Loose Stone, R2200	1	3500	101.0
45	Loose Stone, R2200	1	3500	100.0
46	Loose Stone, R2200	1	3500	99.0
47	Loose Stone, R2200	1	3500	98.0
48	Loose Stone, R2200	1	3500	97.0
49	Loose Stone, R2200	1	3000	100.0
50	Loose Stone, R2200	1	3000	99.0
51	Loose Stone, R2200	1	3000	98.0
52	Loose Stone, R2200	1	3000	97.3
53	Loose Stone, R2200	1	2500	99.0
54	Loose Stone, R2200	1	2500	98.0
55	Loose Stone, R2200	1	2500	96.0
56	Loose Stone, R2200	1	2000	99.0
57	Loose Stone, R2200	1	2000	98.0
58	Loose Stone, R2200	1	2000	97.0
59	Loose Stone, R2200	1	2000	95.4
60	Loose Stone, R2200	1	4000	102.0
61	Loose Stone, R2200	1	4000	101.0
62	Loose Stone, R2200	1	4000	100.0
63	Loose Stone, R2200	1	4000	99.0
64	Loose Stone, R2200	1	4000	98.0
65	Loose Stone, R2200	1	4000	97.7
66	Loose Stone, R2200	1	4500	104.0
67	Loose Stone, R2200	1	4500	103.0
68	Loose Stone, R2200	1	4500	102.0
69	Loose Stone, R2200	1	4500	100.0
70	Loose Stone, R2200	1	4500	99.0
71	Loose Stone, R2200	1	4500	98.2
72	Loose Stone, R2200	1	5000	105.0
73	Loose Stone, R2200	1	5000	104.0
74	Loose Stone, R2200	1	5000	103.0
75	Loose Stone, R2200	1	5000	101.0

Table E1 (Concluded)

Test #	Basin Conditions	Weir Type	Discharge	Tailwater
76	Loose Stone, R2200	1	5000	100.0
77	Loose Stone, R2200	1	5000	99.0
78	Loose Stone, R2200	1	5500	106.0
79	Loose Stone, R2200	1	5500	105.0
80	Loose Stone, R2200	1	5500	103.0
81	Loose Stone, R2200	1	5500	102.0
82	Loose Stone, R2200	1	5500	101.0
83	Loose Stone, R2200		6000	106.0
84	Loose Stone, R2200	1	6000	105.0
85	Loose Stone, R2200	1	6000	104.0
86	Loose Stone, R2200	1	6000	103.0
87	Loose Stone, R2200	1	6500	107.0
88	Loose Stone, R2200	1	6500	106.0
89	Loose Stone, R2200	1	6500	104.0
90	Loose Stone, R2200	1	6500	103.0
91	Loose Stone, R2200	1	7000	107.0
92	Loose Stone, R2200	1	7000	106.0
93	Loose Stone, R2200	1	7000	105.0
94	Loose Stone, R2200	1	7000	104.0
95	Loose Stone, R2200	1	7500	108.0
96	Loose Stone, R2200	1	7500	107.0
97	Loose Stone, R2200	1	7500	104.0
98	Loose Stone, R2200	1	7500	105.0
99	Loose Stone, R2200	1	8000	109.0
100	Loose Stone, R2200		8000	108.0
101	Loose Stone, R2200	1	8000	106.0
102	Loose Stone, R2200	1	8000	105.0
103	Loose Stone, R2200	1	8000	104.0

(Sheet 3 of 3)

Table E2
Large Riprap Stability

Test #	Discharge cfs	Unit q cfs/ft	Critical Yc ft	Normal Yn ft	Tail-water el	Submergence	D50 ft	Movement	q/D50
1	2000	50.0	3.7	6.4	109.5	2.57	1.5	NO	33.33
2	2500	62.5	4.4	7.0	109.5	2.16	1.5	NO	41.67
3	3000	75.0	5.6	7.6	109.5	1.70	1.5	NO	50.00
4	3500	87.5	5.7	9.4	109.5	1.67	1.5	NO	58.33
5	4000	100.0	6.0	10.0	109.5	1.58	1.5	NO	66.67
6	4000	100.0	6.0	10.0	108.0	1.33	1.5	NO	66.67
7	4000	100.0	6.0	10.0	107.0	1.17	1.5	NO	66.67
8	4000	100.0	6.0	10.0	106.0	1.00	1.5	NO	66.67
9	4000	100.0	6.0	10.0	105.0	0.83	1.5	NO	66.67
10	4000	100.0	6.0	10.0	104.0	0.67	1.5	YES	66.67
11	4000	100.0	6.0	10.0	103.0	0.50	1.5	YES	66.67
12	4000	100.0	6.0	10.0	102.0	0.33	1.5	YES	66.67
13	4000	100.0	6.0	10.0	101.0	0.17	1.5	YES	66.67
40	3500	87.5	5.7	9.4	105.0	0.88	2.0	NO	43.75
41	3500	87.5	5.7	9.4	104.0	0.70	2.0	NO	43.75
42	3500	87.5	5.7	9.4	103.0	0.53	2.0	NO	43.75
43	3500	87.5	5.7	9.4	102.0	0.35	2.0	NO	43.75
44	3500	87.5	5.7	9.4	101.0	0.18	2.0	NO	43.75
45	3500	87.5	5.7	9.4	100.0	0.00	2.0	YES	43.75

(Sheet 1 of 4)

Table E2 (Continued)

Test #	Discharge cfs	Unit q cfs/ft	Critical Yc ft	Normal Yn ft	Tail-water el	Submergence	D50 ft	Movement	q/D50
46	3500	87.5	5.7	9.4	99.0	-0.18	2.0	YES	43.75
47	3500	87.5	5.7	9.4	98.0	-0.35	2.0	YES	43.75
48	3500	87.5	5.7	9.4	97.0	-0.53	2.0	YES	43.75
49	3000	75.0	5.6	7.6	100.0	0.00	2.0	NO	37.50
50	3000	75.0	5.6	7.6	99.0	-0.18	2.0	YES	37.50
51	3000	75.0	5.6	7.6	98.0	-0.36	2.0	YES	37.50
52	3000	75.0	5.6	7.6	97.3	-0.48	2.0	YES	37.50
53	2500	62.5	4.4	7.0	99.0	-0.23	2.0	NO	31.25
54	2500	62.5	4.4	7.0	98.0	-0.45	2.0	YES	31.25
55	2500	62.5	4.4	7.0	98.0	-0.91	2.0	YES	31.25
56	2000	50.0	3.7	6.4	99.0	-0.27	2.0	NO	25.00
57	2000	50.0	3.7	6.4	98.0	-0.54	2.0	NO	25.00
58	2000	50.0	3.7	6.4	97.0	-0.81	2.0	YES	25.00
59	2000	50.0	3.7	6.4	95.4	-1.24	2.0	YES	25.00
60	4000	100.0	6.0	10.0	102.0	0.33	2.0	NO	50.00
61	4000	100.0	6.0	10.0	101.0	0.17	2.0	YES	50.00
62	4000	100.0	6.0	10.0	100.0	0.00	2.0	YES	50.00
63	4000	100.0	6.0	10.0	99.0	-0.17	2.0	YES	50.00
64	4000	100.0	6.0	10.0	98.0	-0.33	2.0	YES	50.00
65	4000	100.0	6.0	10.0	97.7	-0.38	2.0	YES	50.00
66	4500	112.5	6.4	10.8	104.0	0.63	2.0	NO	56.25

(Sheet 2 of 4)

Table E2 (Continued)

Test #	Discharge cfs	Unit q cfs/ft	Critical Yc ft	Normal Yn ft	Tail-water el	Submergence	D50 ft	Movement	q/D50
67	4500	112.5	6.4	10.8	103.0	0.47	2.0	YES	56.25
68	4500	112.5	6.4	10.8	102.0	0.31	2.0	YES	56.25
69	4500	112.5	6.4	10.8	100.0	0.00	2.0	YES	56.25
70	4500	112.5	6.4	10.8	99.0	-0.16	2.0	YES	56.25
71	4500	112.5	6.4	10.8	98.2	-0.28	2.0	YES	56.25
72	5000	125.0	6.8	11.6	105.0	0.74	2.0	NO	62.50
73	5000	125.0	6.8	11.6	104.0	0.59	2.0	YES	62.50
74	5000	125.0	6.8	11.6	103.0	0.44	2.0	YES	62.50
75	5000	125.0	6.8	11.6	101.0	0.15	2.0	YES	62.50
76	5000	125.0	6.8	11.6	100.0	0.00	2.0	YES	62.50
77	5000	125.0	6.8	11.6	99.0	-0.15	2.0	YES	62.50
78	5500	137.5	7.2	12.0	106.0	0.83	2.0	NO	68.75
79	5500	137.5	7.2	12.0	105.0	0.69	2.0	YES	68.75
80	5500	137.5	7.2	12.0	103.0	0.42	2.0	YES	68.75
81	5500	137.5	7.2	12.0	102.0	0.28	2.0	YES	68.75
82	5500	137.5	7.2	12.0	101.0	0.14	2.0	YES	68.75
83	6000	150.0	7.6	12.8	106.0	0.79	2.0	NO	75.00
84	6000	150.0	7.6	12.8	105.0	0.66	2.0	YES	75.00
85	6000	150.0	7.6	12.8	104.0	0.53	2.0	YES	75.00

(Sheet 3 of 4)

Table E2 (Concluded)

Test #	Discharge cfs	Unit q cfs/ft	Critical Yc ft	Normal Yn ft	Tail-water el	Submergence	D50 ft	Movement	q/D50
86	6000	150.0	7.6	12.8	103.0	0.39	2.0	YES	75.00
87	6500	162.5	8.0	13.6	107.0	0.68	2.0	NO	81.25
88	6500	162.5	8.0	13.6	106.0	0.75	2.0	YES	81.25
89	6500	162.5	8.0	13.6	104.0	0.50	2.0	YES	81.25
90	6500	162.5	8.0	13.6	103.0	0.38	2.0	YES	81.25
91	7000	175.0	8.4	14.0	107.0	0.83	2.0	NO	87.50
92	7000	175.0	8.4	14.0	106.0	0.71	2.0	YES	87.50
93	7000	175.0	8.4	14.0	105.0	0.60	2.0	YES	87.50
94	7000	175.0	8.4	14.0	104.0	0.48	2.0	YES	87.50
95	7500	187.5	8.5	15.2	108.0	0.94	2.0	NO	93.75
96	7500	187.5	8.5	15.2	107.0	0.82	2.0	YES	93.75
97	7500	187.5	8.5	15.2	104.0	0.47	2.0	YES	93.75
98	7500	187.5	8.5	15.2	105.0	0.59	2.0	YES	93.75
99	8000	200.0	8.8	15.8	109.0	1.02	2.0	NO	100.00
100	8000	200.0	8.8	15.8	108.0	0.91	2.0	YES	100.00
101	8000	200.0	8.8	15.8	106.0	0.68	2.0	YES	100.00
102	8000	200.0	8.8	15.8	105.0	0.57	2.0	YES	100.00
103	8000	200.0	8.8	15.8	104.0	0.45	2.0	YES	100.00

Table E3
Small Riprap Stability

Test #	Discharge cfs	Unit q cfs/ft	Critical Yc ft	Normal Yn ft	Tailwater El	Submergence	D50 ft	Movement	q/D50
40	3500	87.5	5.7	9.4	105.0	0.88	0.75	NO	116.67
41	3500	87.5	5.7	9.4	104.0	0.70	0.75	NO	116.67
42	3500	87.5	5.7	9.4	103.0	0.53	0.75	NO	116.67
43	3500	87.5	5.7	9.4	102.0	0.35	0.75	NO	116.67
44	3500	87.5	5.7	9.4	101.0	0.18	0.75	NO	116.67
45	3500	87.5	5.7	9.4	100.0	0.00	0.75	NO	116.67
46	3500	87.5	5.7	9.4	99.0	-0.18	0.75	NO	116.67
47	3500	87.5	5.7	9.4	98.0	-0.35	0.75	NO	116.67
48	3500	87.5	5.7	9.4	97.0	-0.53	0.75	NO	116.67
49	3000	75.0	5.6	7.6	100.0	0.00	0.75	NO	100.00
50	3000	75.0	5.6	7.6	99.0	-0.18	0.75	NO	100.00
51	3000	75.0	5.6	7.6	98.0	-0.36	0.75	NO	100.00
52	3000	75.0	5.6	7.6	97.3	-0.48	0.75	NO	100.00
53	2500	62.5	4.4	7.0	99.0	-0.23	0.75	NO	83.33
54	2500	62.5	4.4	7.0	98.0	-0.45	0.75	NO	83.33
55	2500	62.5	4.4	7.0	96.0	-0.91	0.75	NO	83.33
56	2000	50.0	3.7	6.4	99.0	-0.27	0.75	NO	66.67
57	2000	50.0	3.7	6.4	98.0	-0.54	0.75	NO	66.67
58	2000	50.0	3.7	6.4	97.0	-0.61	0.75	NO	66.67

(Sheet 1 of 4)

Table E3 (Continued)

Test #	Discharge cfs	Unit q cfs/ft	Critical Yc ft	Normal Yn ft	Tailwater El	Submergence	D50 ft	Movement	q/D50
59	2000	50.0	3.7	6.4	95.4	-1.24	0.75	NO	66.67
60	4000	100.0	6.0	10.0	102.0	0.33	0.75	NO	133.33
61	4000	100.0	6.0	10.0	101.0	0.17	0.75	YES	133.33
62	4000	100.0	6.0	10.0	100.0	0.00	0.75	NO	133.33
63	4000	100.0	6.0	10.0	99.0	-0.17	0.75	NO	133.33
64	4000	100.0	6.0	10.0	98.0	-0.33	0.75	NO	133.33
65	4000	100.0	6.0	10.0	97.7	-0.38	0.75	NO	133.33
66	4500	112.5	6.4	10.8	104.0	0.63	0.75	YES	150.00
67	4500	112.5	6.4	10.8	103.0	0.47	0.75	YES	150.00
68	4500	112.5	6.4	10.8	102.0	0.31	0.75	YES	150.00
69	4500	112.5	6.4	10.8	100.0	0.00	0.75	YES	150.00
70	4500	112.5	6.4	10.8	99.0	-0.16	0.75	YES	150.00
71	4500	112.5	6.4	10.8	98.2	-0.28	0.75	YES	150.00
72	5000	125.0	6.8	11.6	105.0	0.74	0.75	NO	166.67
73	5000	125.0	6.8	11.6	104.0	0.59	0.75	YES	166.67
74	5000	125.0	6.8	11.6	103.0	0.44	0.75	YES	166.67
75	5000	125.0	6.8	11.6	101.0	0.15	0.75	YES	166.67
76	5000	125.0	6.8	11.6	100.0	0.00	0.75	YES	166.67
77	5000	125.0	7.2	12.0	99.0	-0.14	0.75	YES	166.67
78	5500	137.5	7.2	12.0	106.0	0.83	0.75	NO	163.33

Table E3 (Continued)

Test #	Discharge cfs	Unit q cfs/ft	Critical Yc ft	Normal Yn ft	Tailwater El	Submergence	D50 ft	Movement	q/D50
79	5500	137.5	7.2	12.0	105.0	0.69	0.75	YES	183.33
80	5500	137.5	7.2	12.0	103.0	0.42	0.75	YES	183.33
81	5500	137.5	7.2	12.0	102.0	0.28	0.75	YES	183.33
82	5500	137.5	7.2	12.0	101.0	0.14	0.75	YES	183.33
83	6000	150.0	7.6	12.8	106.0	0.79	0.75	NO	200.00
84	6000	150.0	7.6	12.8	105.0	0.66	0.75	YES	200.00
85	6000	150.0	7.6	12.8	104.0	0.53	0.75	YES	200.00
86	6000	150.0	7.6	12.8	103.0	0.39	0.75	YES	200.00
87	6500	162.5	8.0	13.6	107.0	0.88	0.75	NO	216.67
88	6500	162.5	8.0	13.6	106.0	0.75	0.75	YES	216.67
89	6500	162.5	8.0	13.6	104.0	0.50	0.75	YES	216.67
90	6500	162.5	8.0	13.6	103.0	0.38	0.75	YES	216.67
91	7000	175.0	8.4	14.0	107.0	0.83	0.75	NO	233.33
92	7000	175.0	8.4	14.0	106.0	0.71	0.75	YES	233.33
93	7000	175.0	8.4	14.0	105.0	0.60	0.75	YES	233.33
94	7000	175.0	8.4	14.0	104.0	0.48	0.75	YES	233.33
95	7500	187.5	8.5	15.2	106.0	0.94	0.75	NO	250.00
96	7500	187.5	8.5	15.2	107.0	0.82	0.75	YES	250.0
97	7500	187.5	8.5	15.2	104.0	0.47	0.75	YES	250.0
98	7500	187.5	8.5	15.2	105.0	0.59	0.75	YES	266.67
99	8000	200.0	8.8	15.8	109.0	1.02	0.75	NO	266.67

(Sheet 3 of 4)

Table E3 (Concluded)

Test #	Discharge cfs	Unit q cfs/ft	Critical Yc ft	Normal Yn ft	Tailwater El	Submergence	D50 ft	Movement	q/D50
100	8000	200.0	8.8	15.8	108.0	0.91	0.75	YES	266.67
101	8000	200.0	8.8	15.8	106.0	0.68	0.75	YES	266.67
102	8000	200.0	8.8	15.8	105.0	0.57	0.75	YES	266.67
103	8000	200.0	8.8	15.8	104.0	0.45	0.75	YES	266.67

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PLAN

DEC
RIPRAP SIZE AND LOCATION
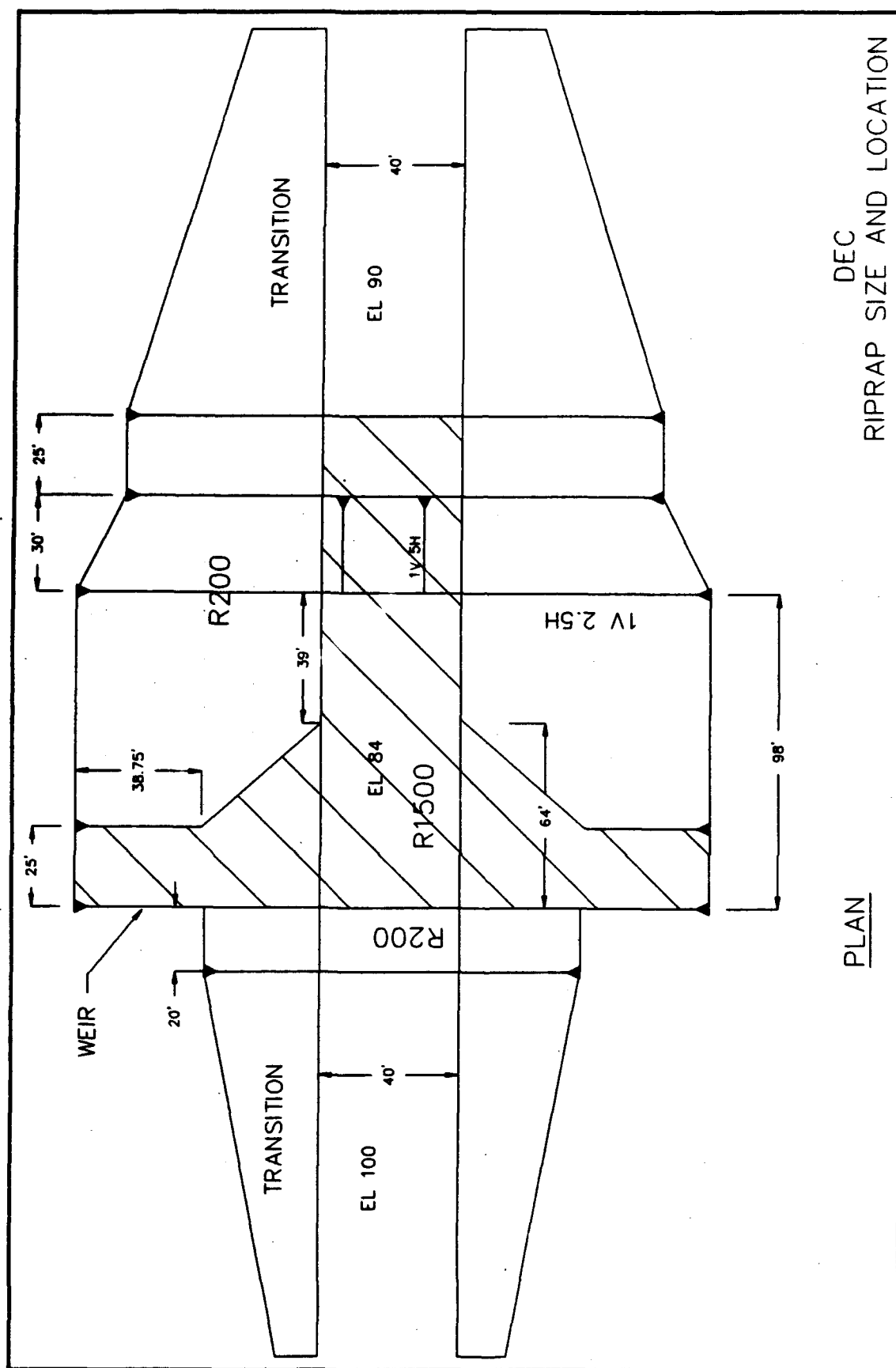
Figure E1. Drop structure plan view

PROFILE

DEC
RIPRAP SECTION
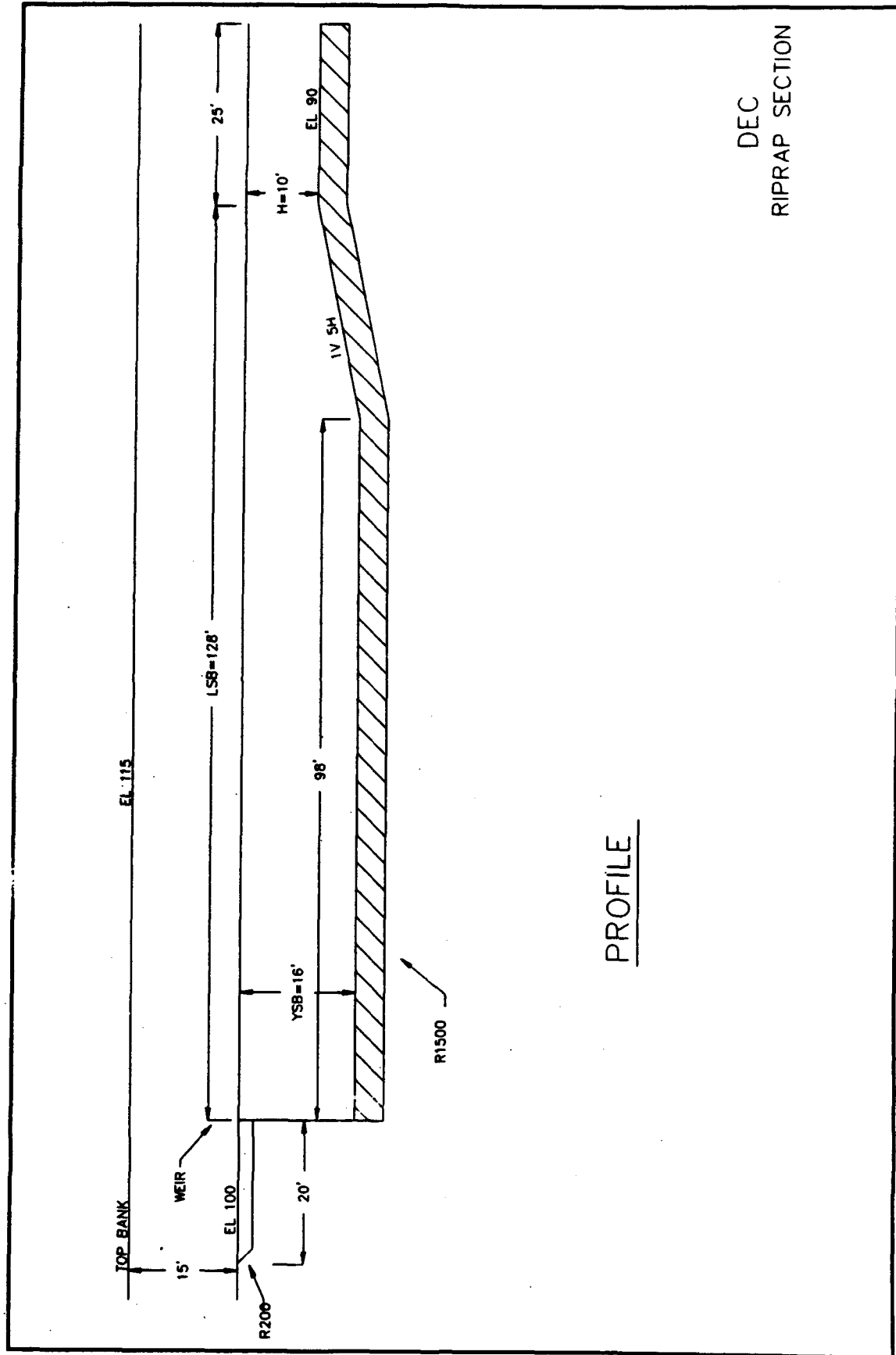
Figure E2. Drop stucture profile
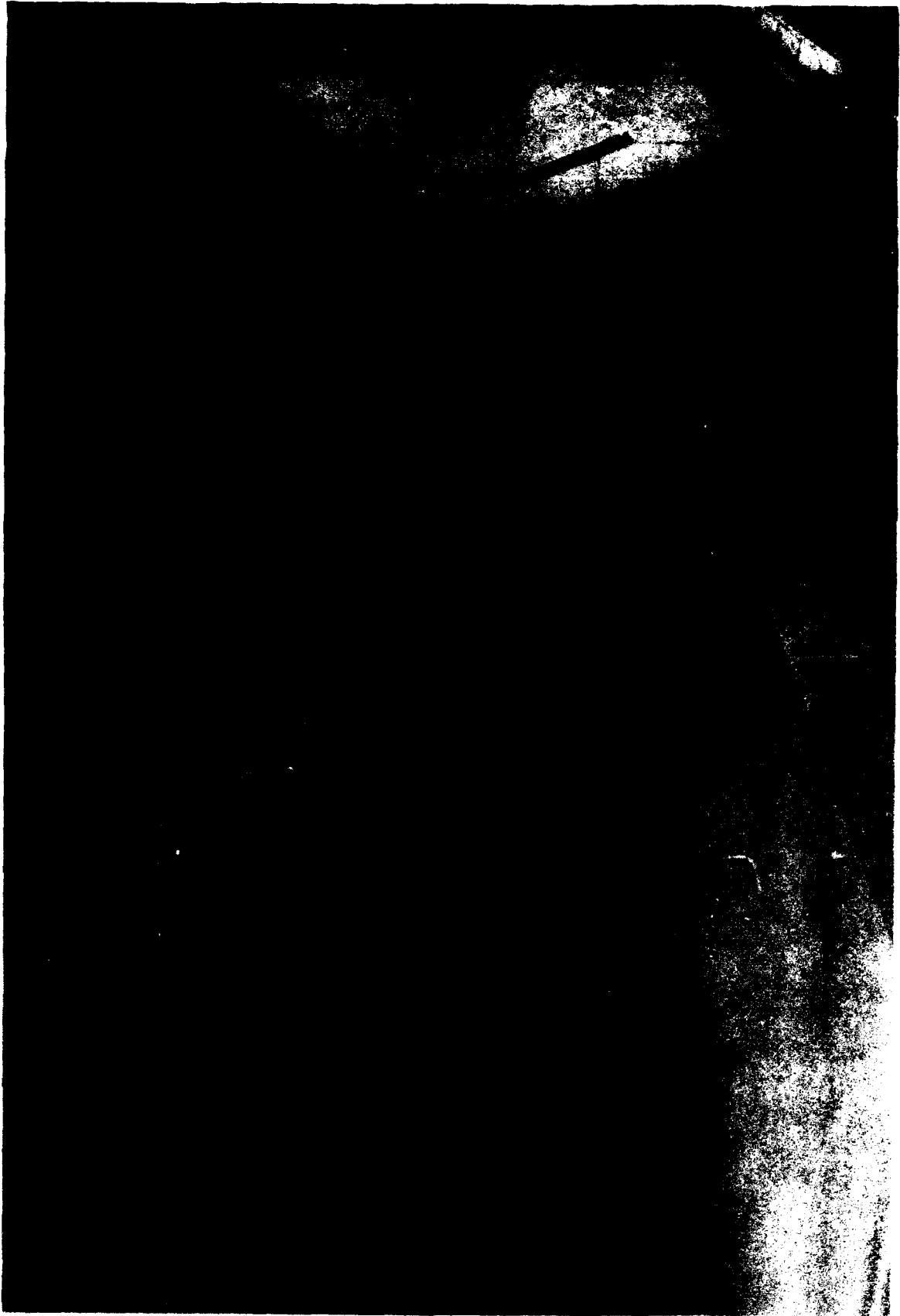
Figure E3. Looking upstream, Type 1 weir, R1500 stone
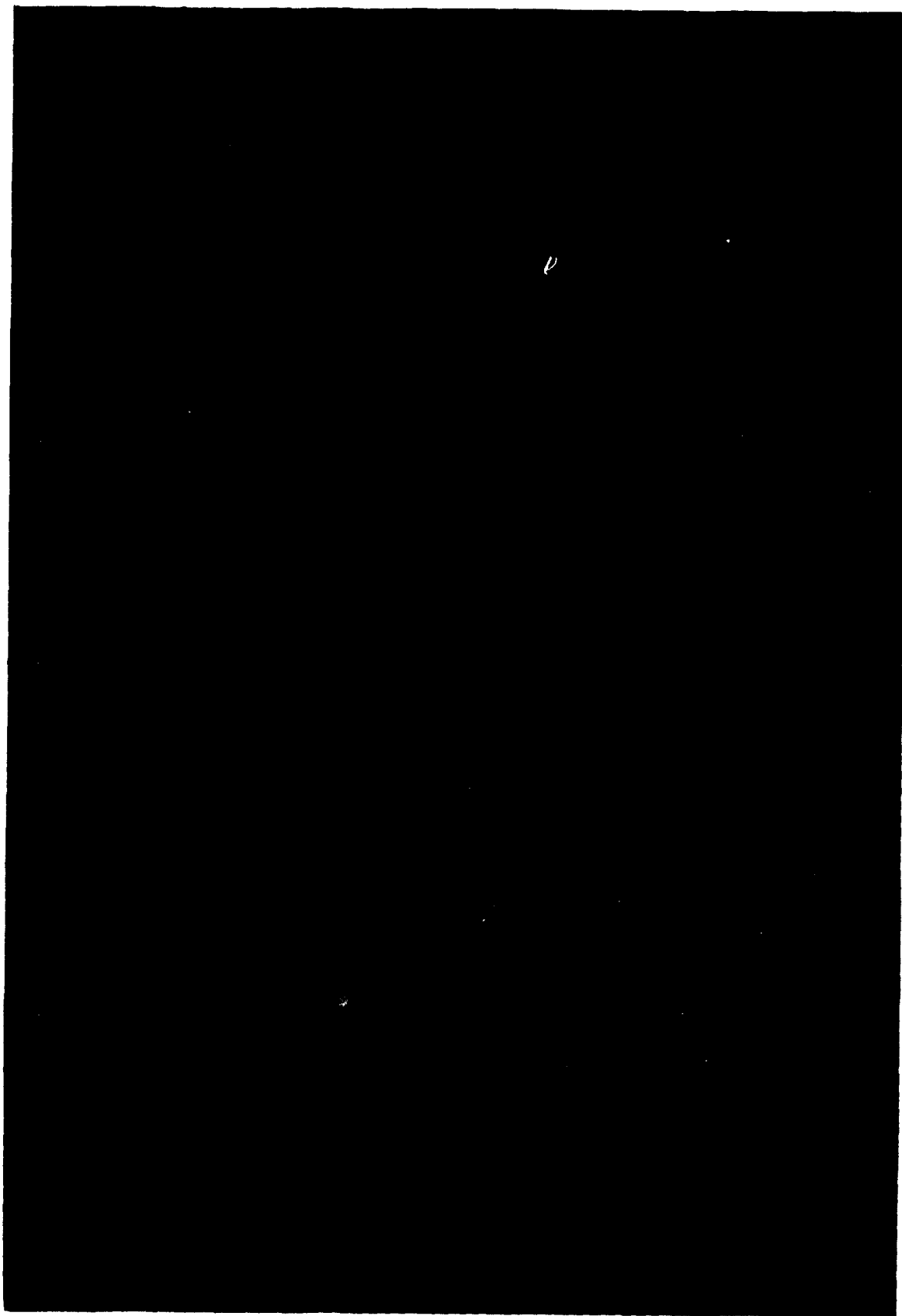
Figure E4. Looking downstream, Type 1 weir, R1500 stone
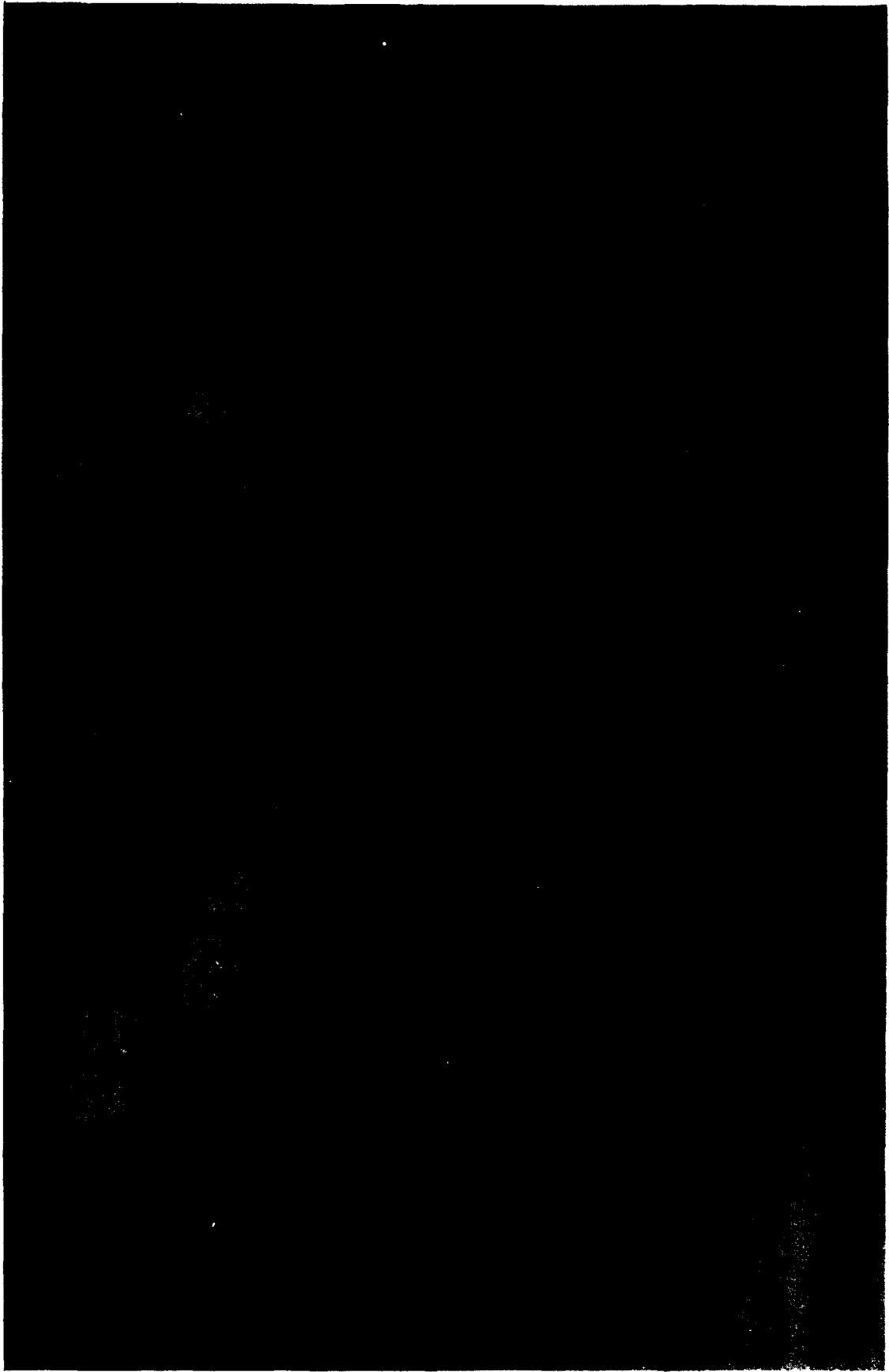
Figure E5. Upstream view of test area, Type 1 weir, R1500 stone
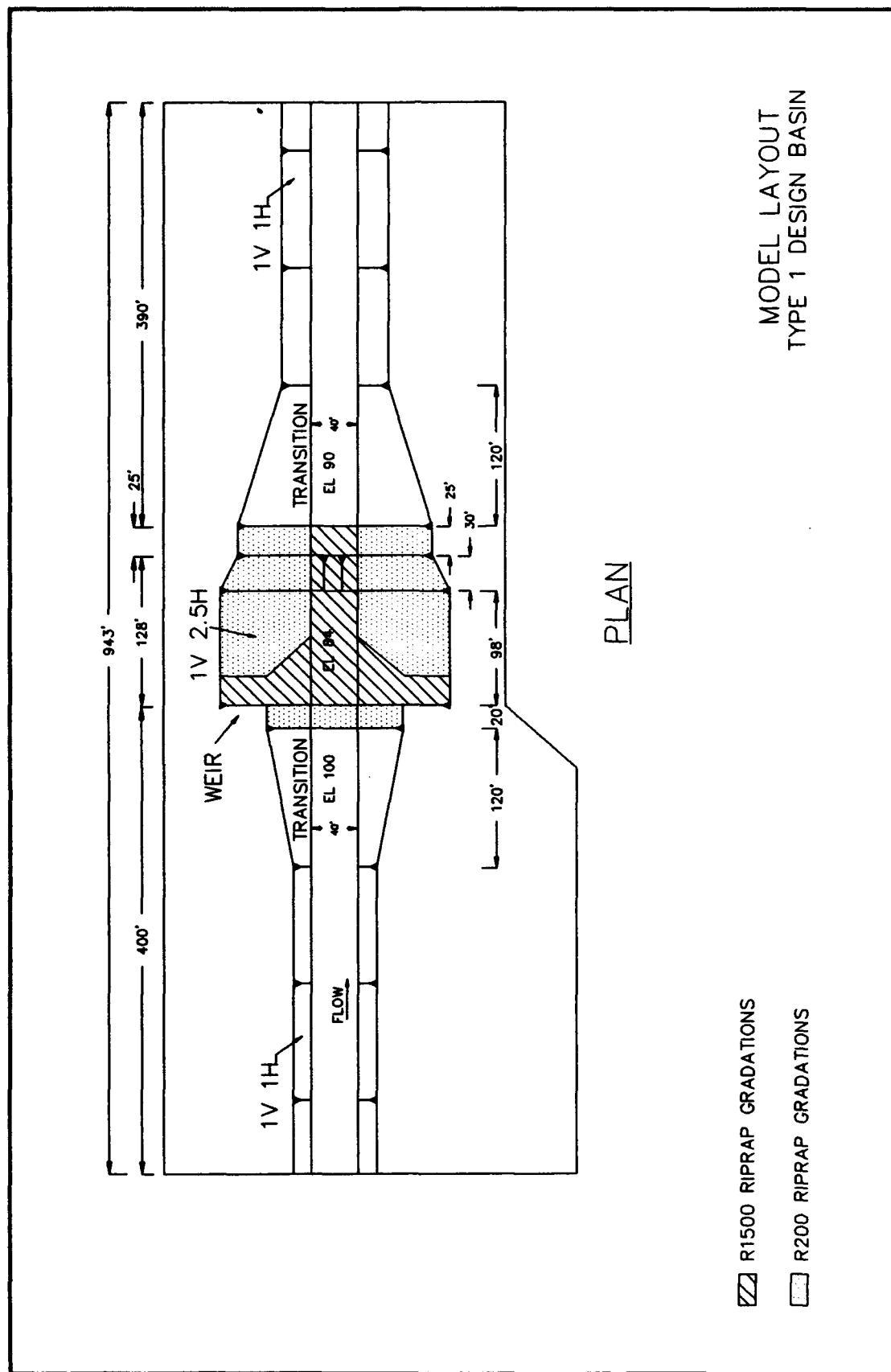
Figure E6. Model layout in prototype dimensions

PLAN

PIEZOMETER LOCATIONS
VELOCITY LOCATIONS
TYPE 1 DESIGN BASIN

TRANSITION

EL 90

273'   213'   153'

WEIR

TRANSITION

EL 100

EL 84

20' 0'   80'   140'   260'

* PIEZOMETER LOCATIONS

⊠ VELOCITY LOCATIONS
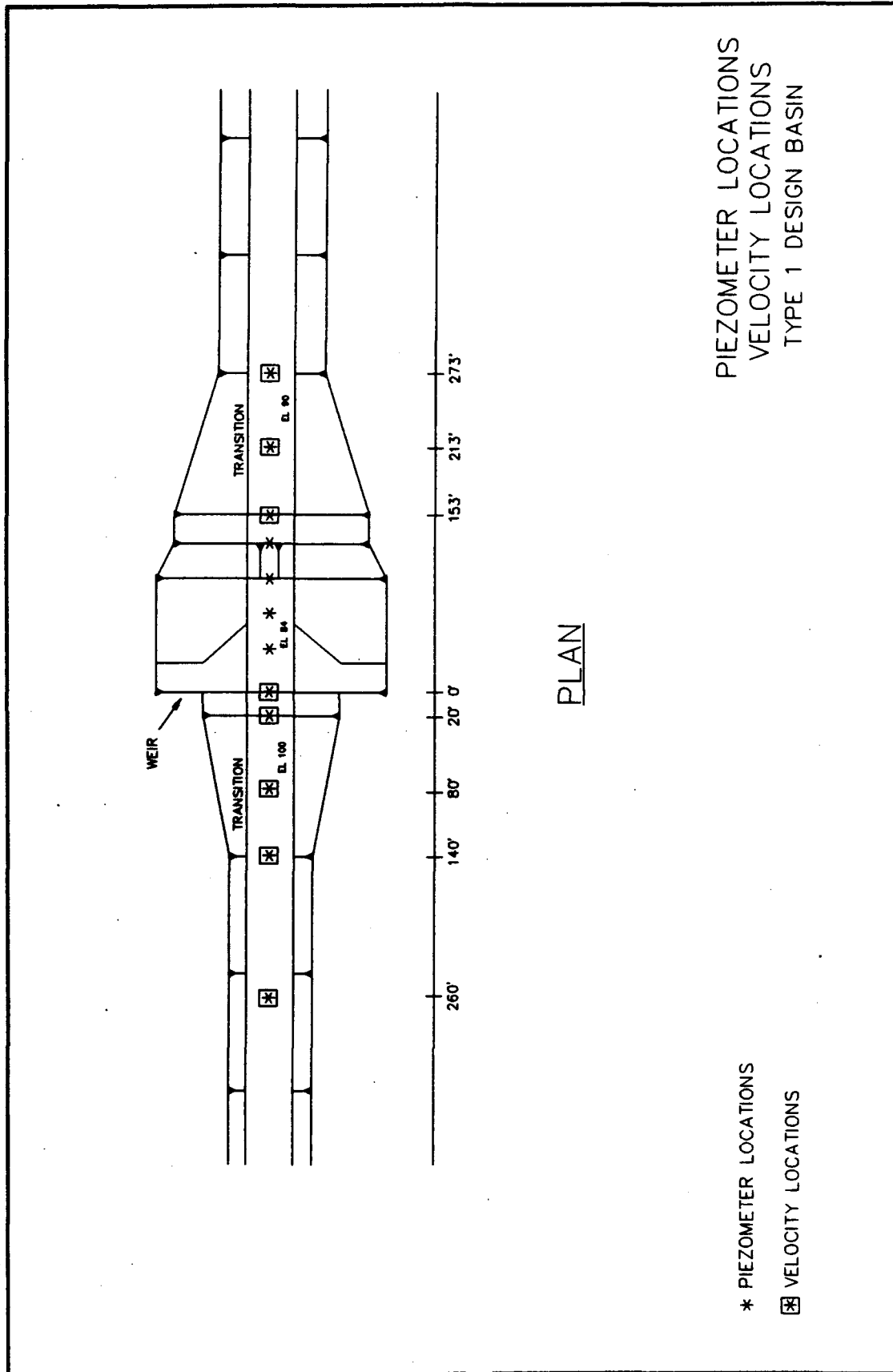
Figure E7. Piezometer and velocity measurement locations

EL 115

40'

EL 100

EL 84

2.5

1

TYPE 1 DESIGN WEIR

EL 115

40'

EL 100

12'

EL 112

EL 84

TYPE 2 DESIGN WEIR

WEIR CROSS-SECTIONAL SHAPES
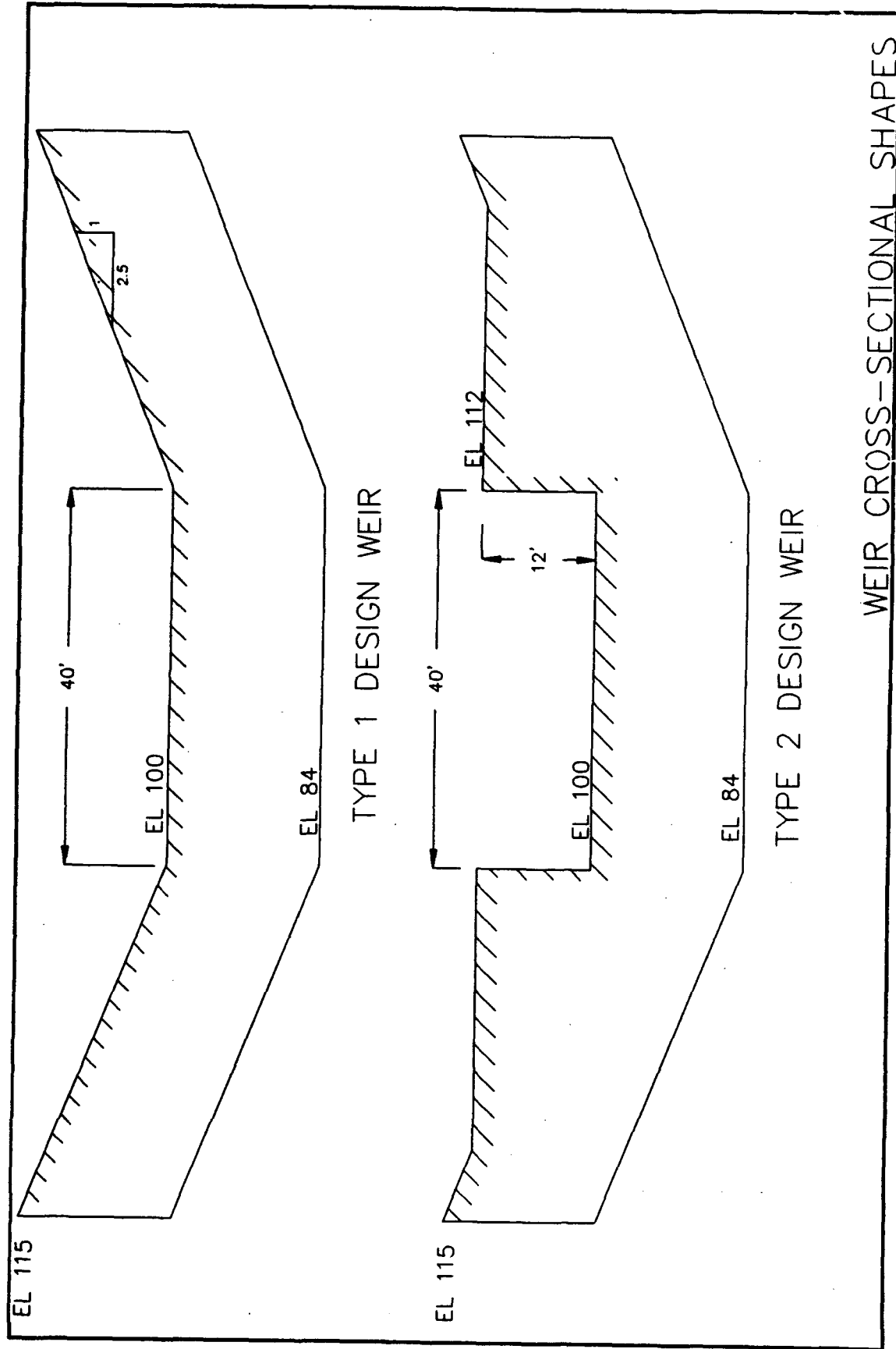
Figure E8. Weir cross-sectional shapes
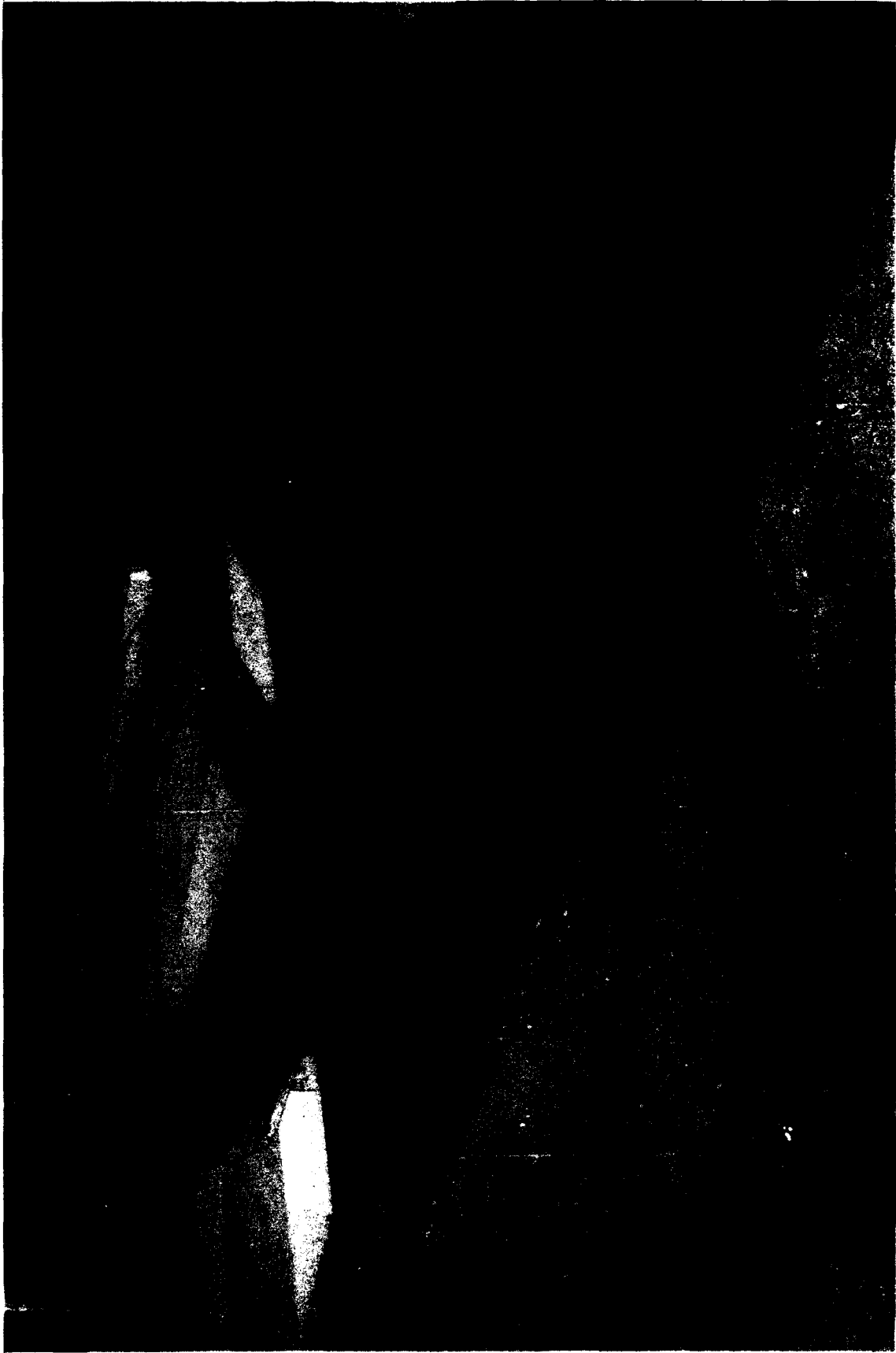
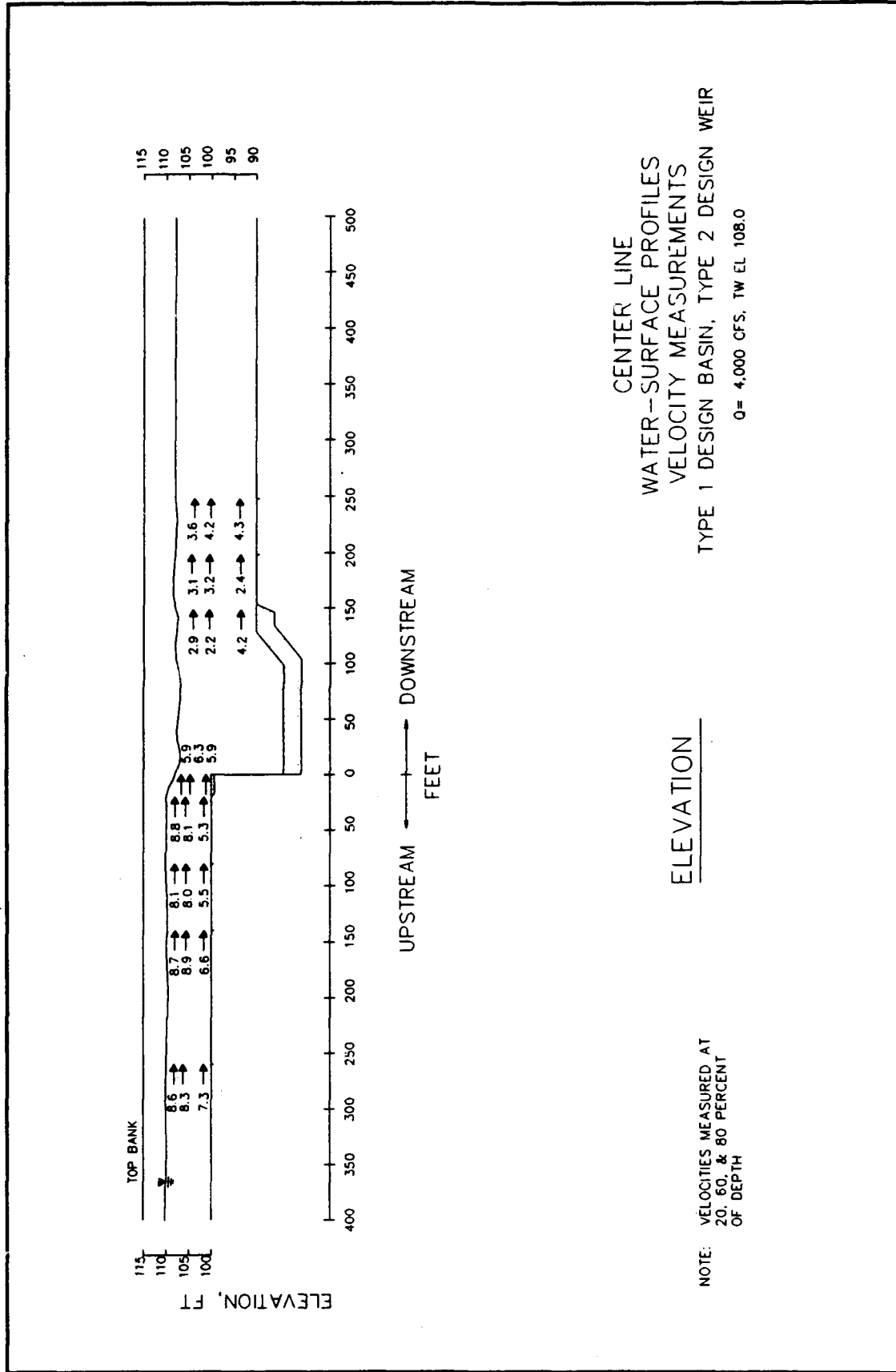Figure E9. Type 1 weir, Q = 2000 cfs, TW el = 109.5
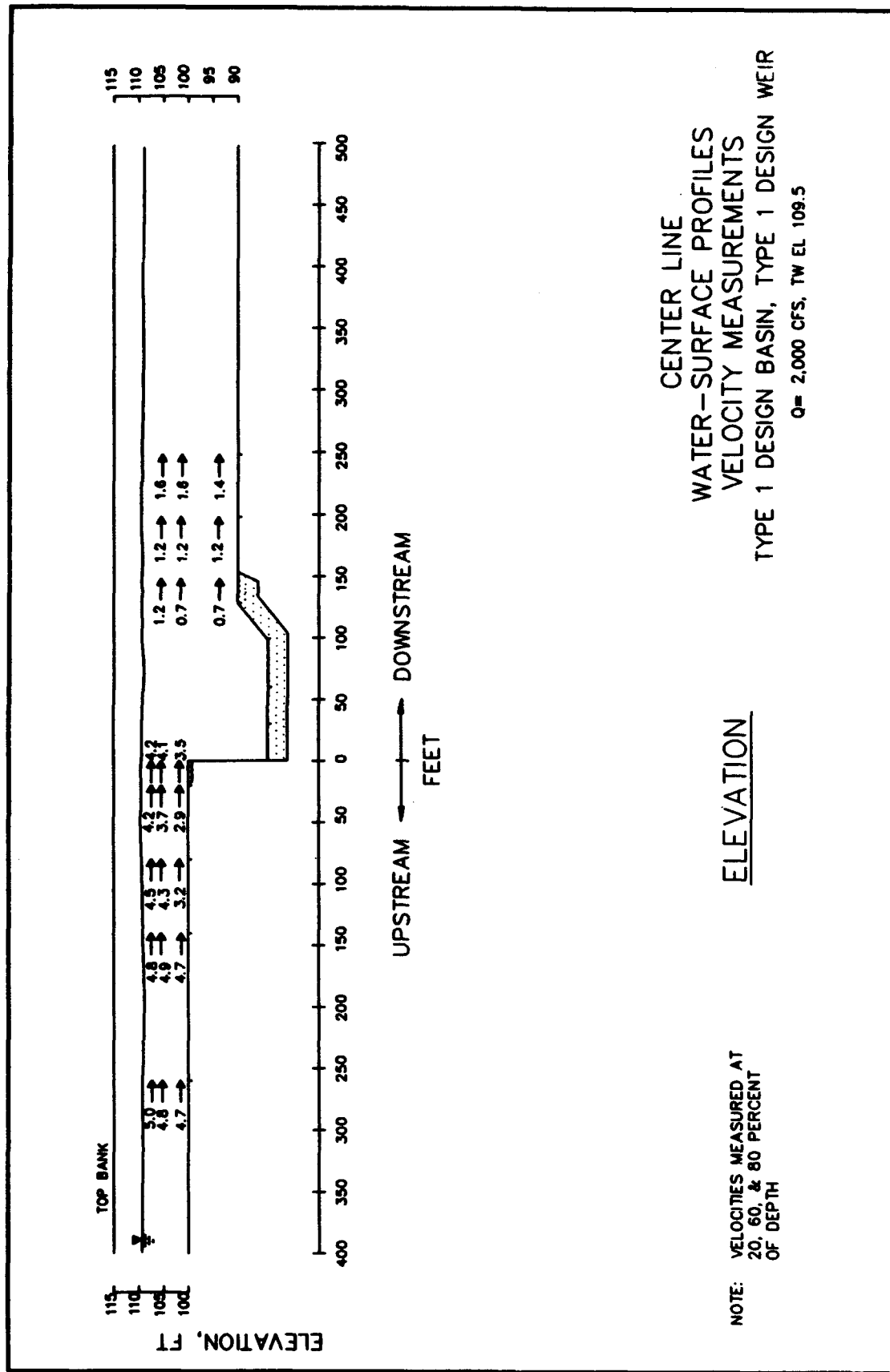
Figure E10. Water-surface profile and velocity measurements, Type 1 weir, Q = 2000 cfs, TW el = 109.5

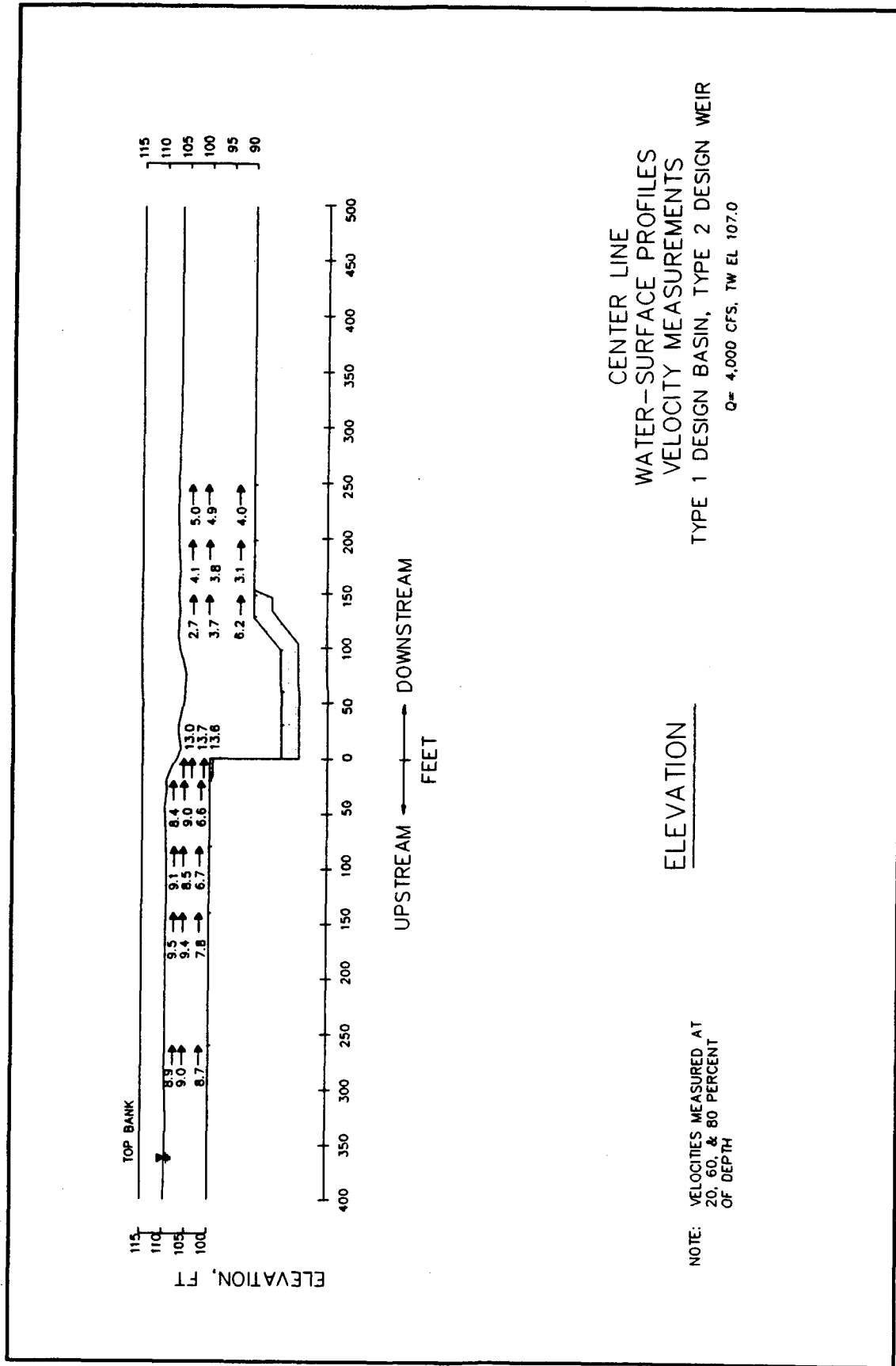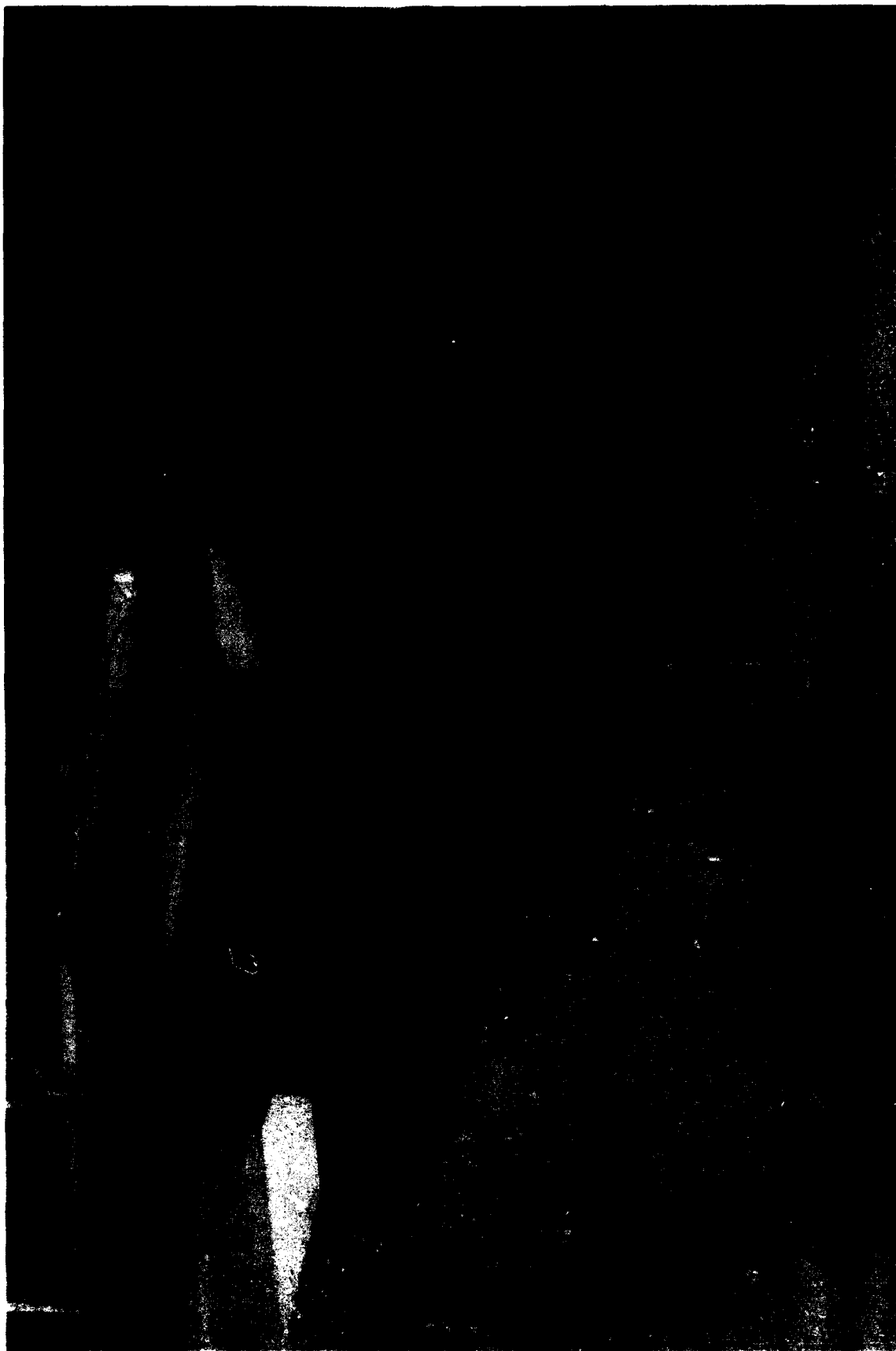Figure E11. Water-surface profile and velocity measurements, Type 1 weir, Q = 2500 cfs, TW el = 109.5
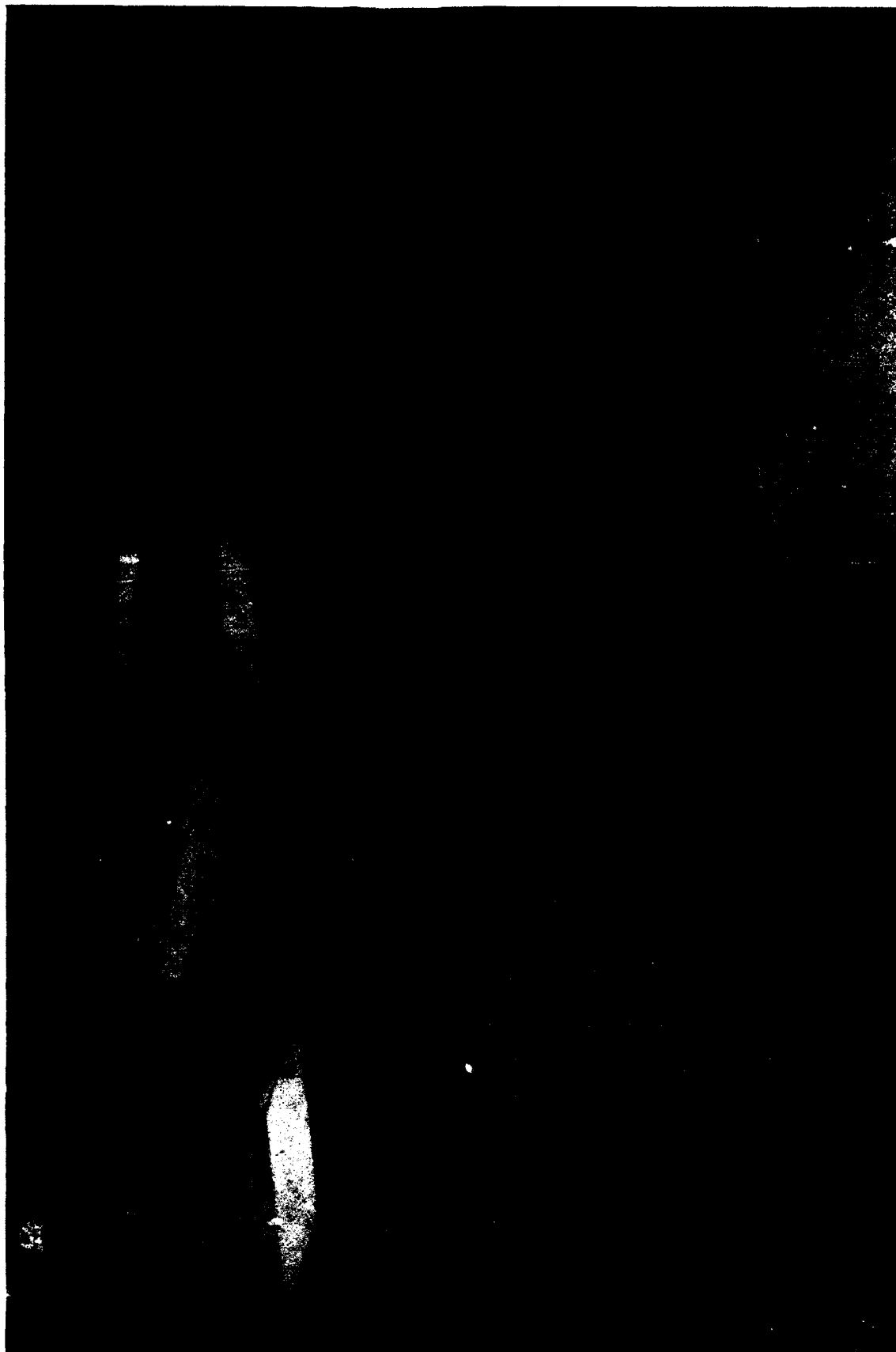
Figure E12. Type 1 weir, Q = 3000 cfs, TW el = 109.5
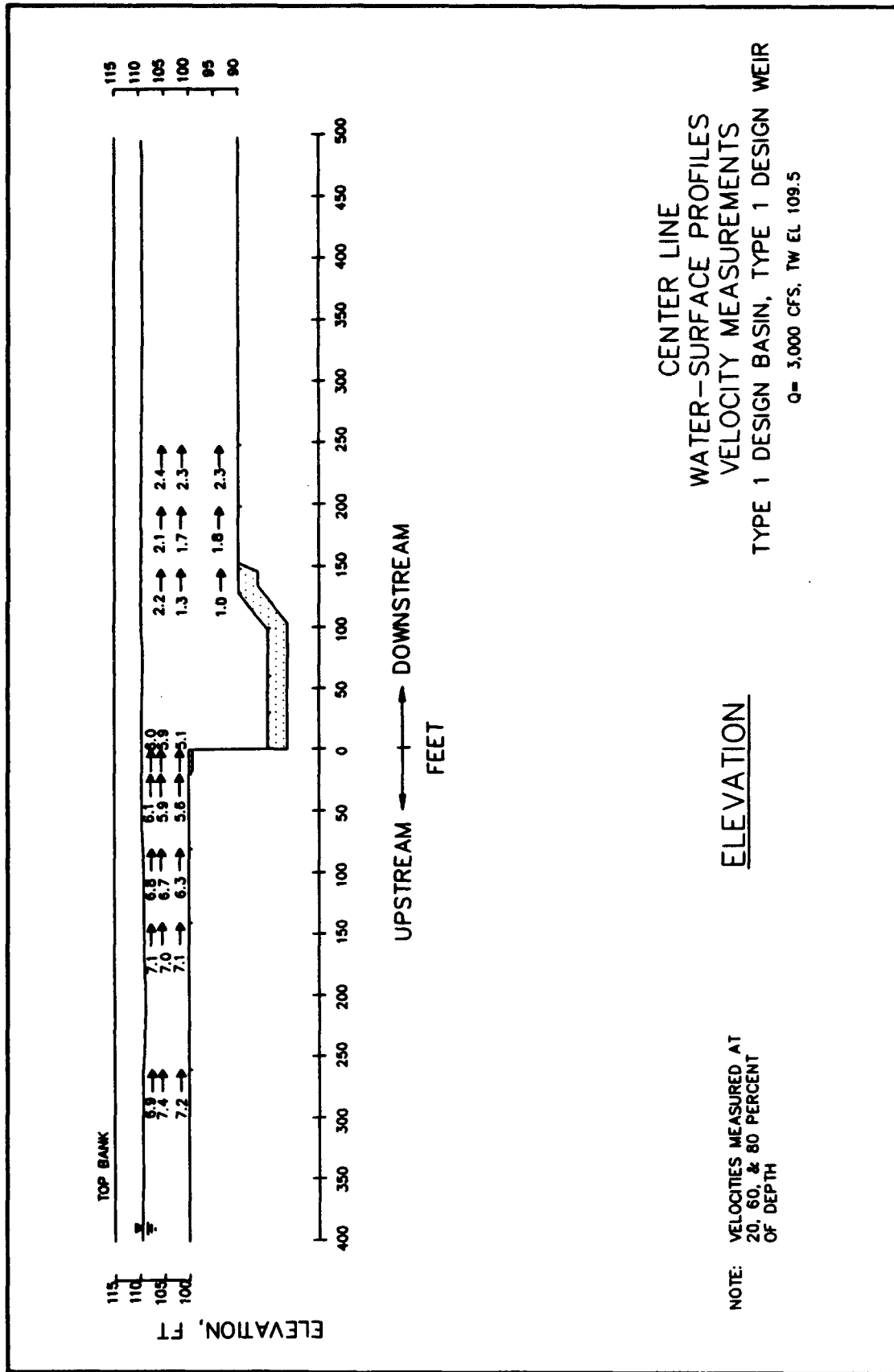
Figure E13. Water-surface profile and velocity measurements, Type 1 weir, Q = 3000 cfs, TW el = 109.5
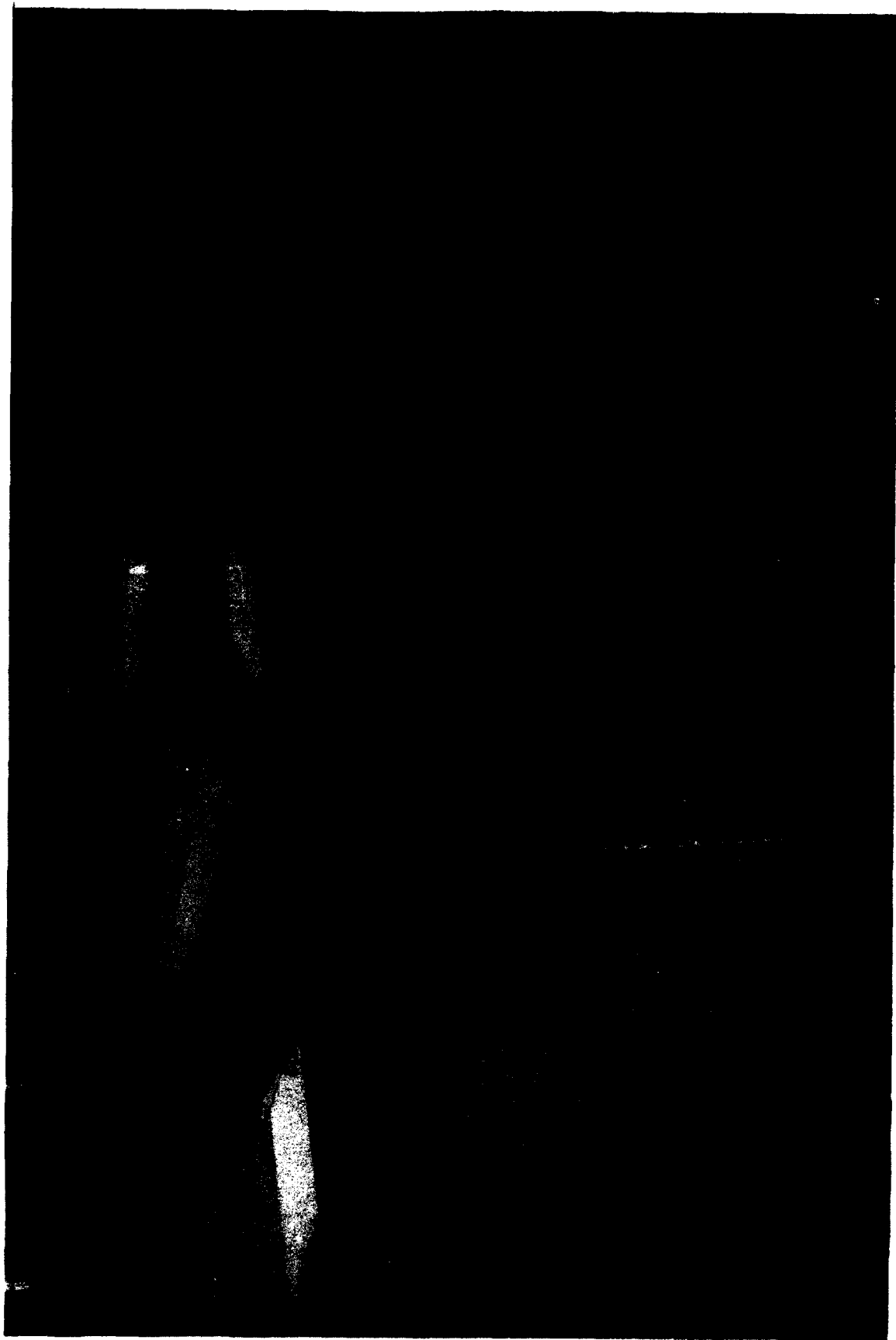
Figure E14. Type 1 weir, Q = 4000 cfs, TW el = 109.5
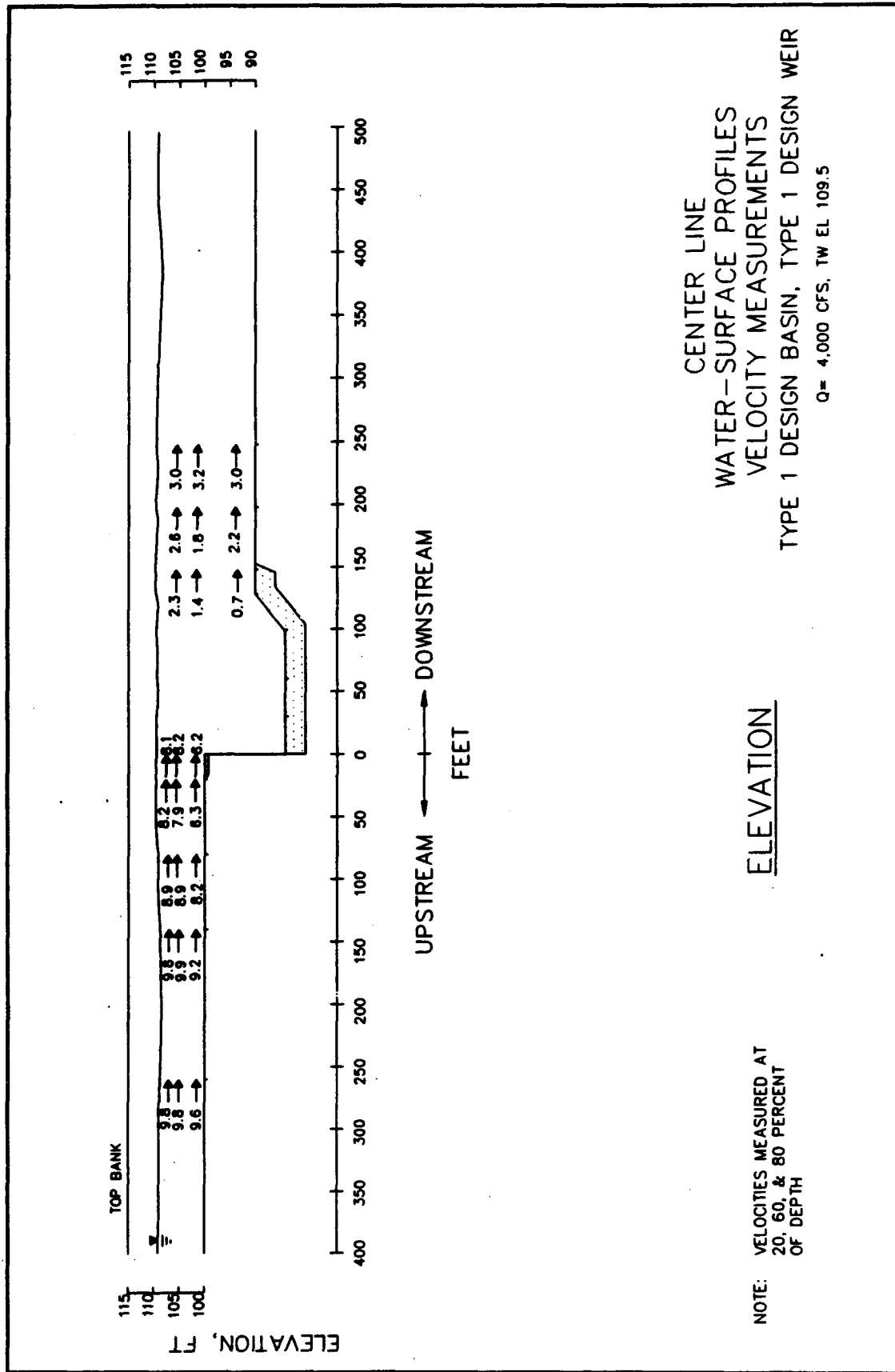
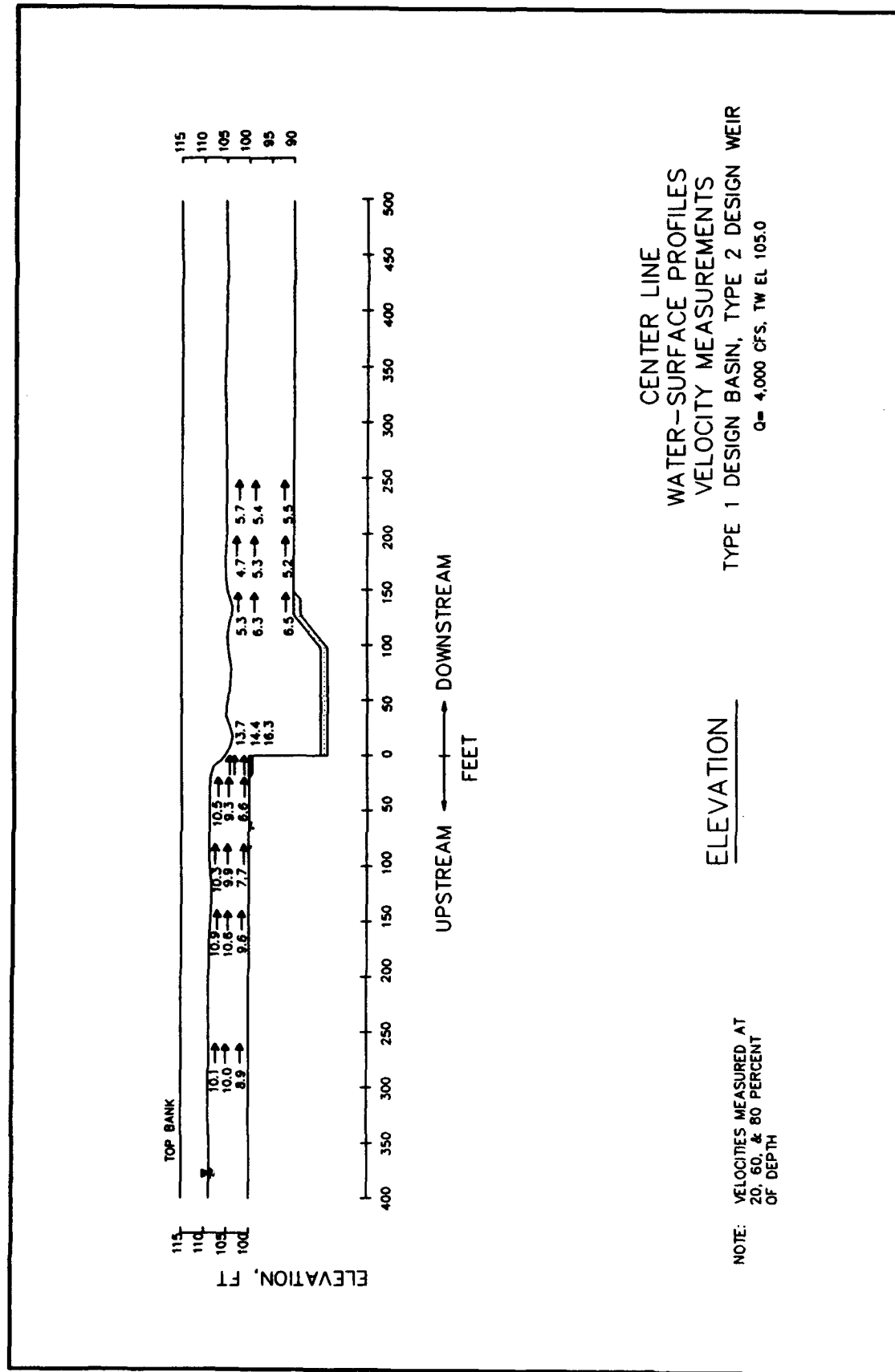Figure E15. Water-surface profile and velocity measurements, Type 1 weir, Q = 4000 cfs, TW el = 109.5
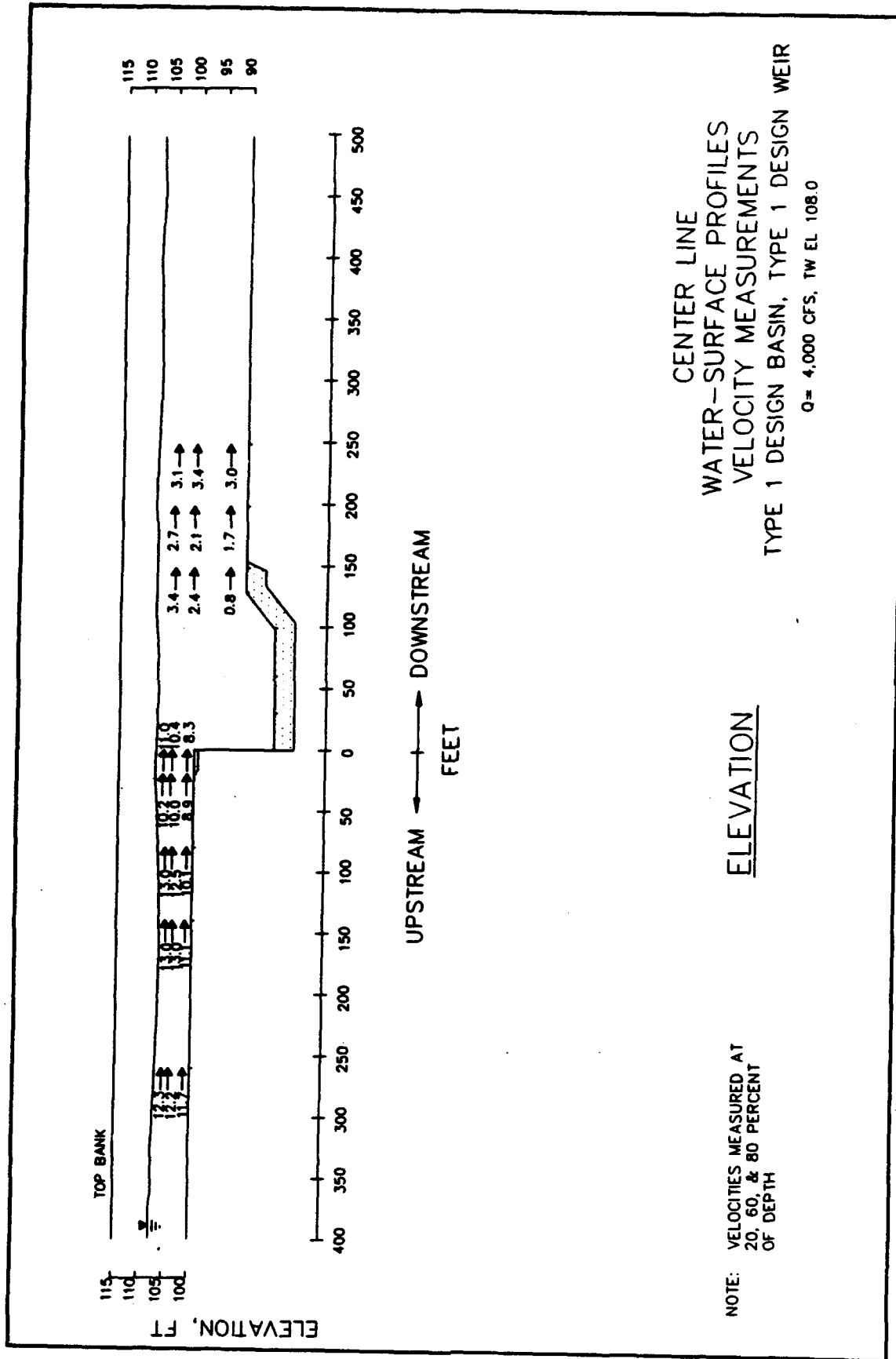
Figure E16. Water-surface profile and velocity measurements, Type 1 weir, Q = 4000 cfs, TW el = 108.0
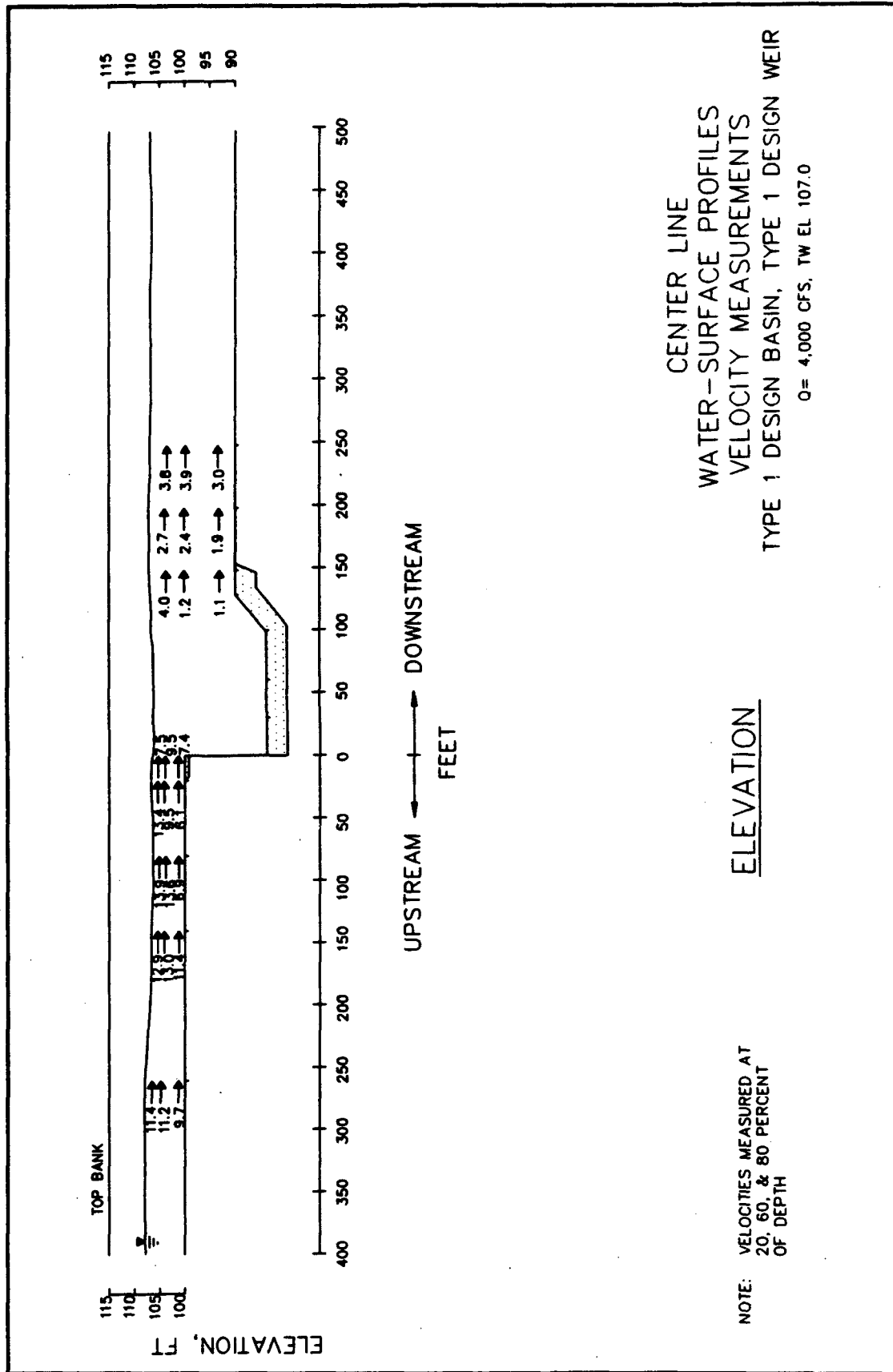
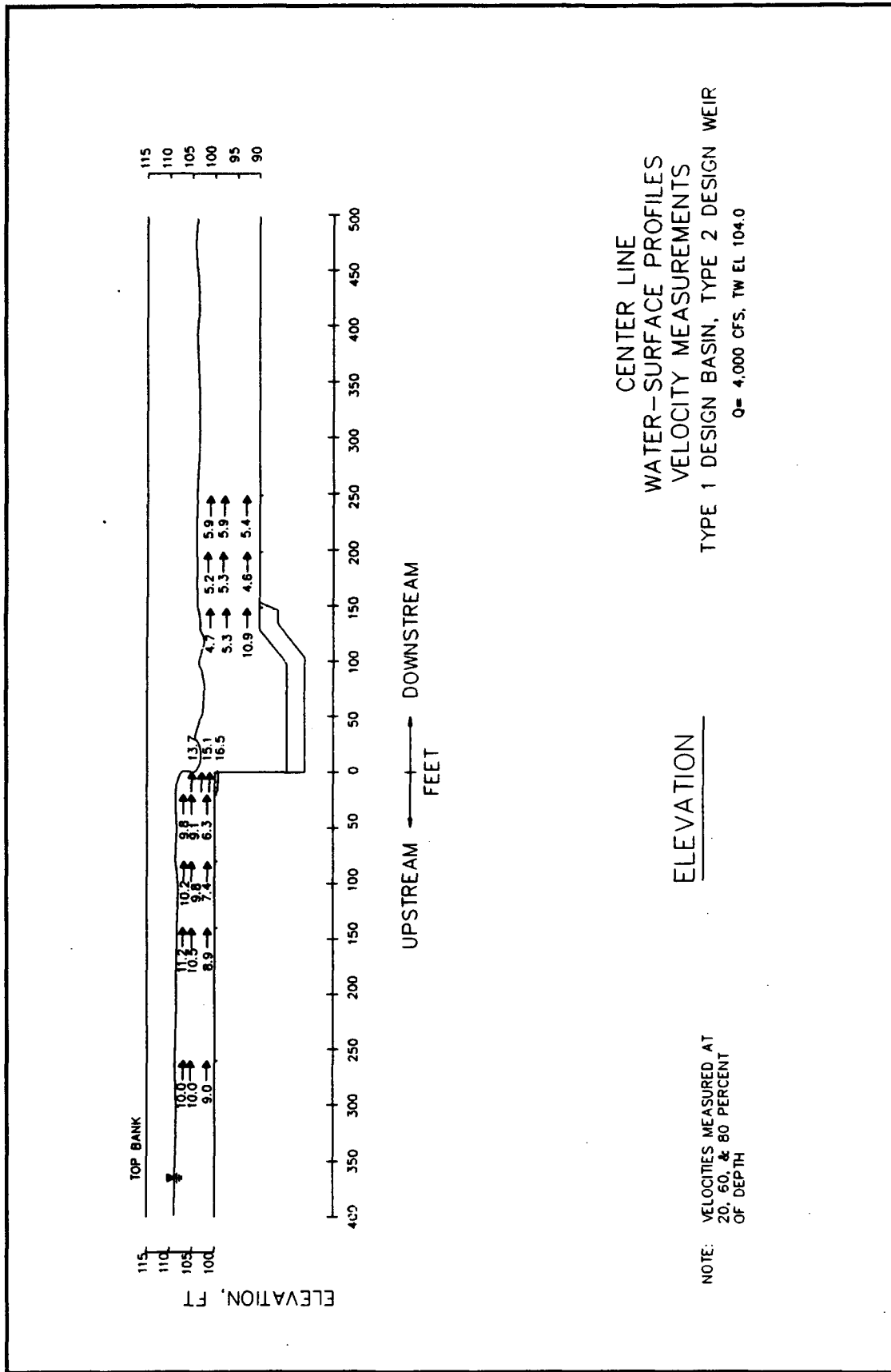Figure E17. Water-surface profile and velocity measurements, Type 1 weir, Q = 4000 cfs, TW el = 107.0
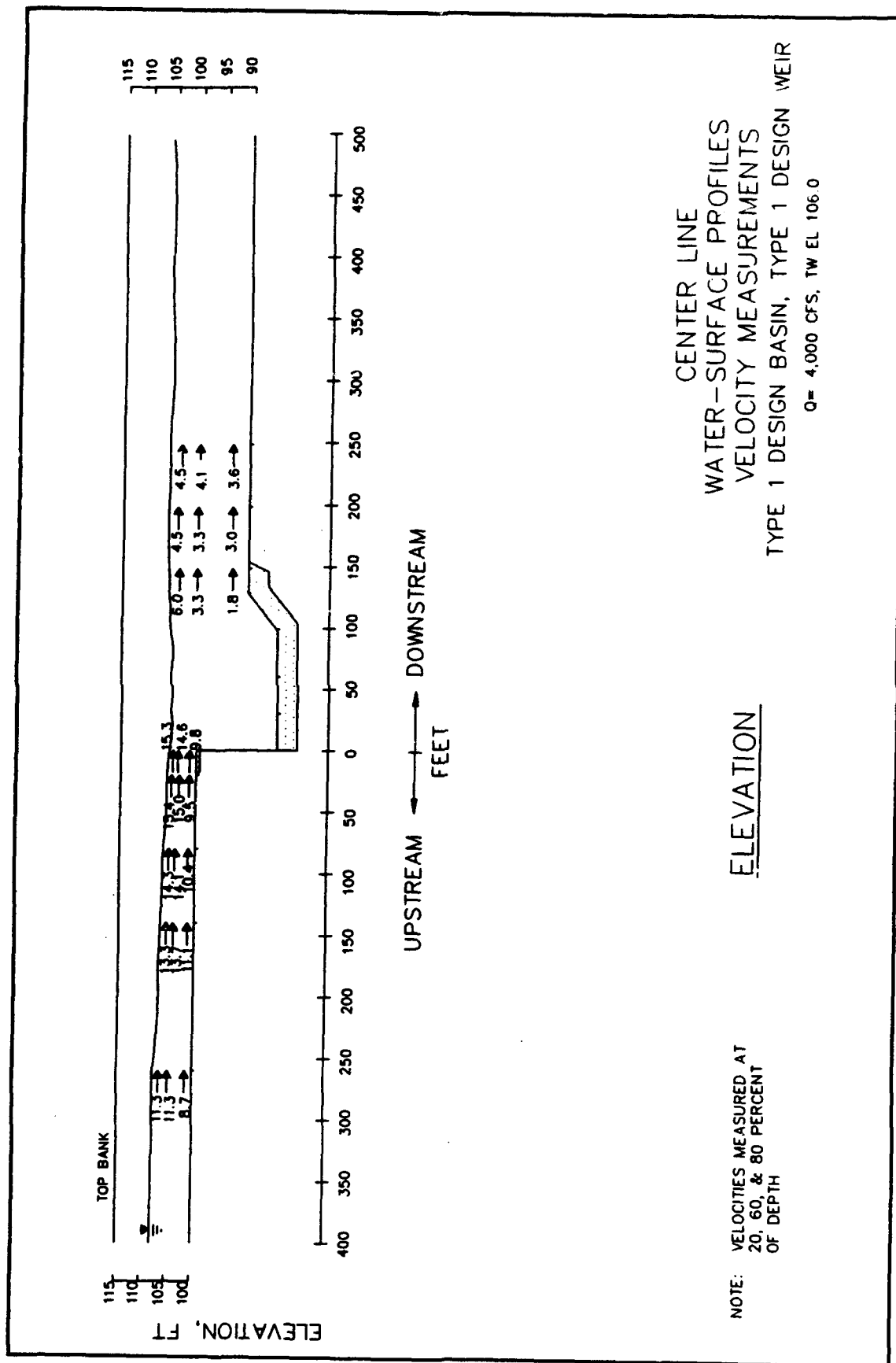
Figure E18. Water-surface profile and velocity measurements, Type 1 weir, Q = 4000 cfs, TW el = 106.0
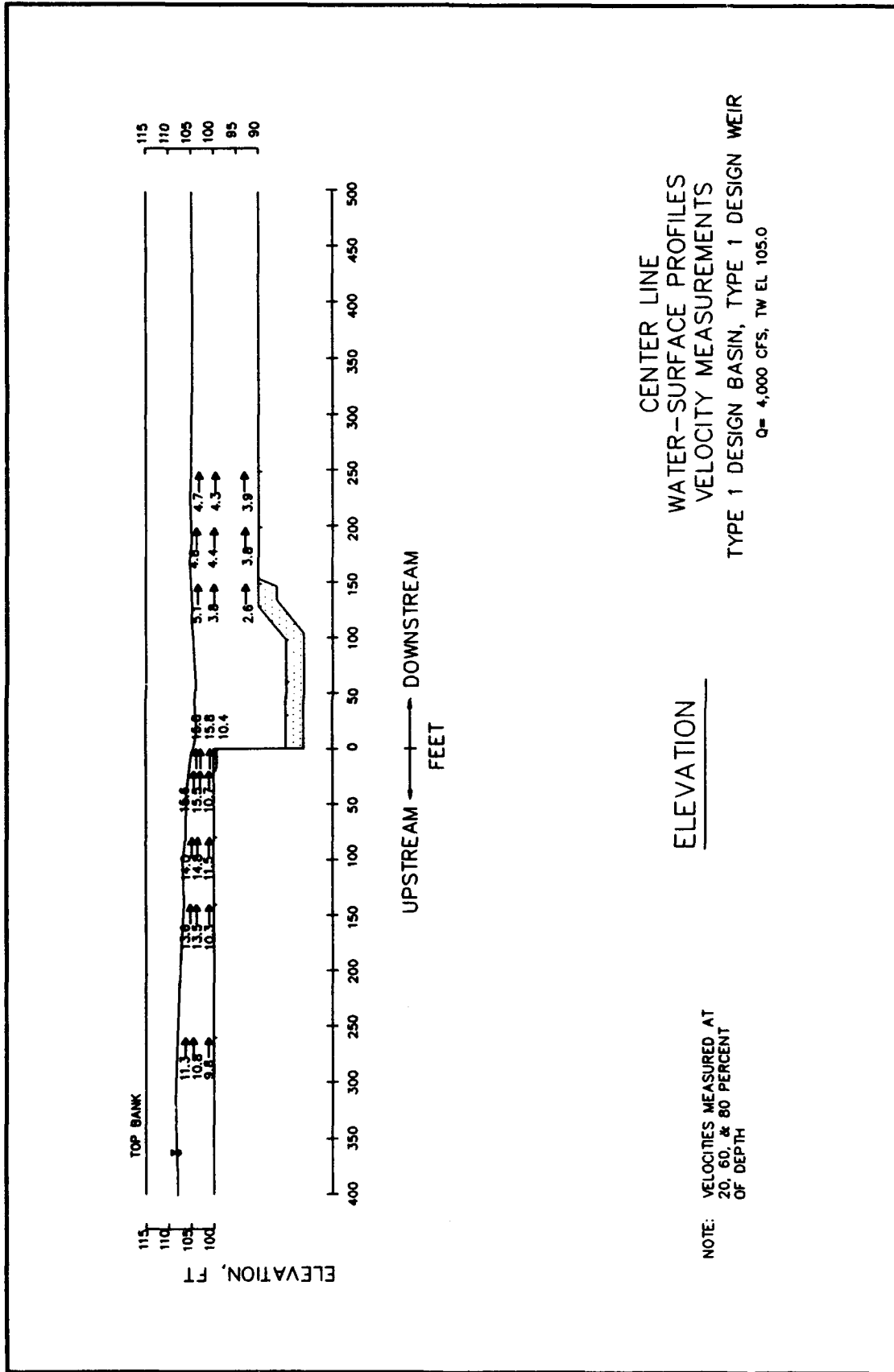
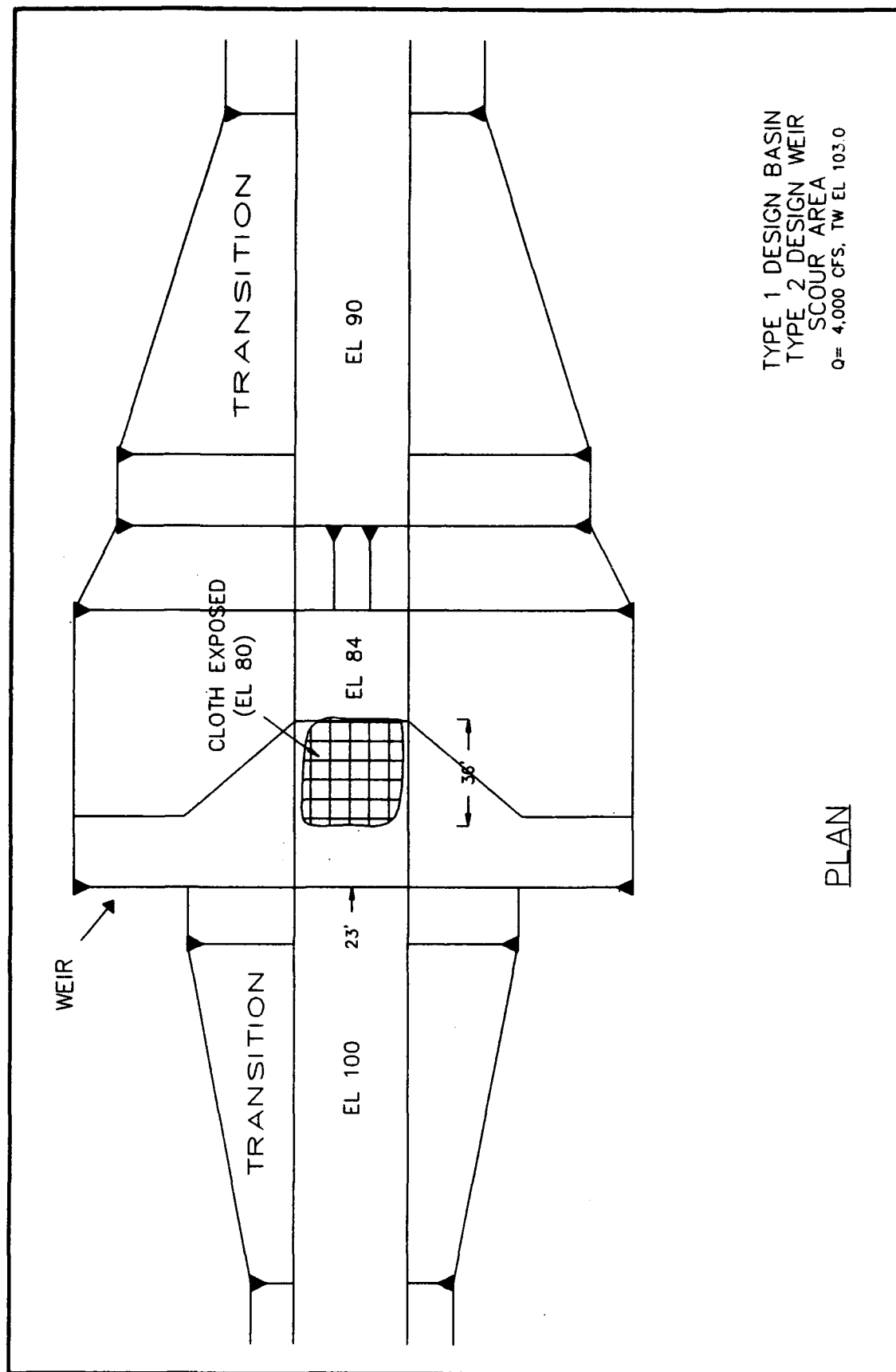Figure E19. Water-surface profile and velocity measurements, Type 1 weir, Q = 4000 cfs, TW el = 105.0
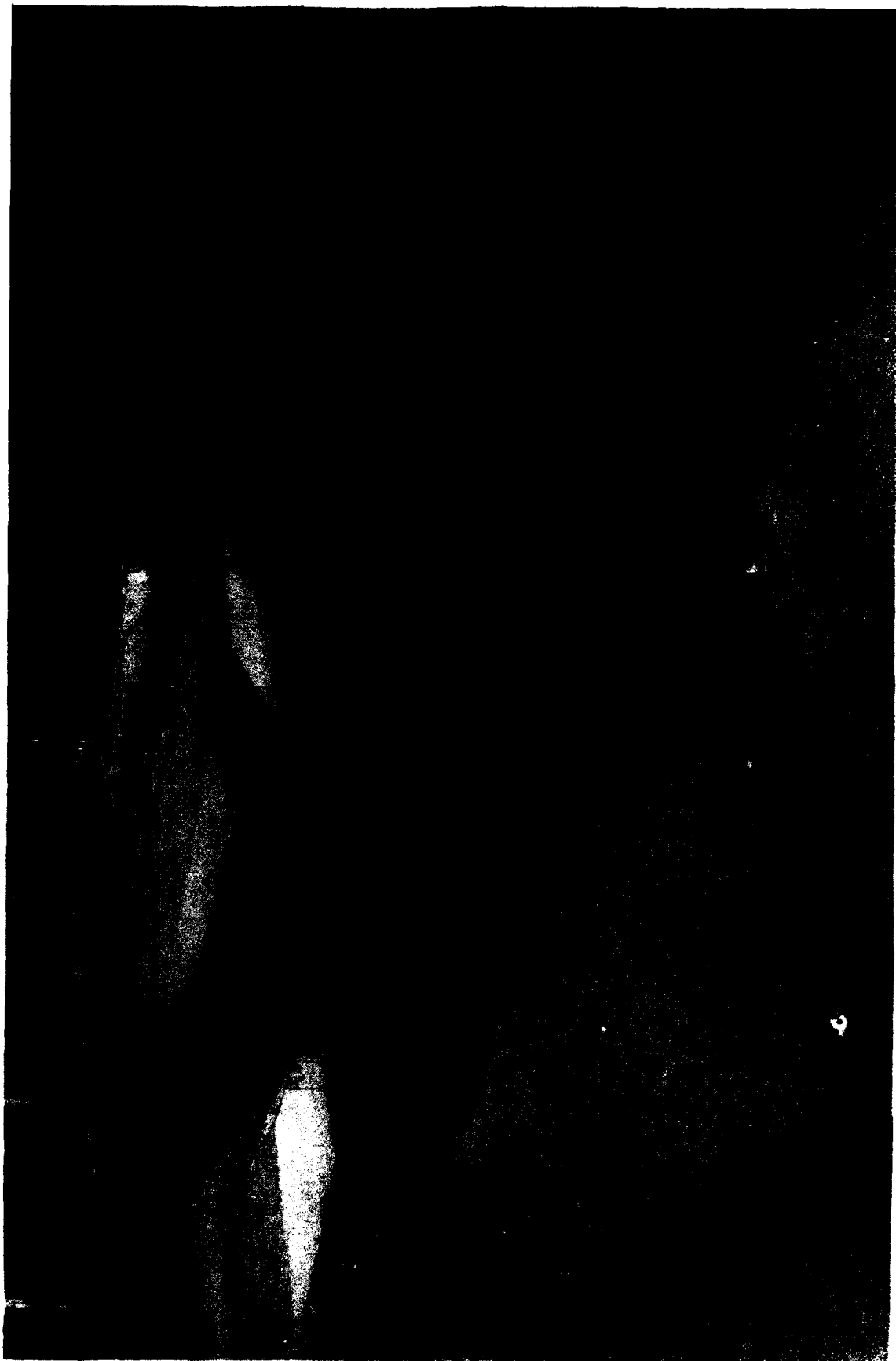
Figure E20. Type 1 weir, Q = 4000 cfs, TW el = 105.0
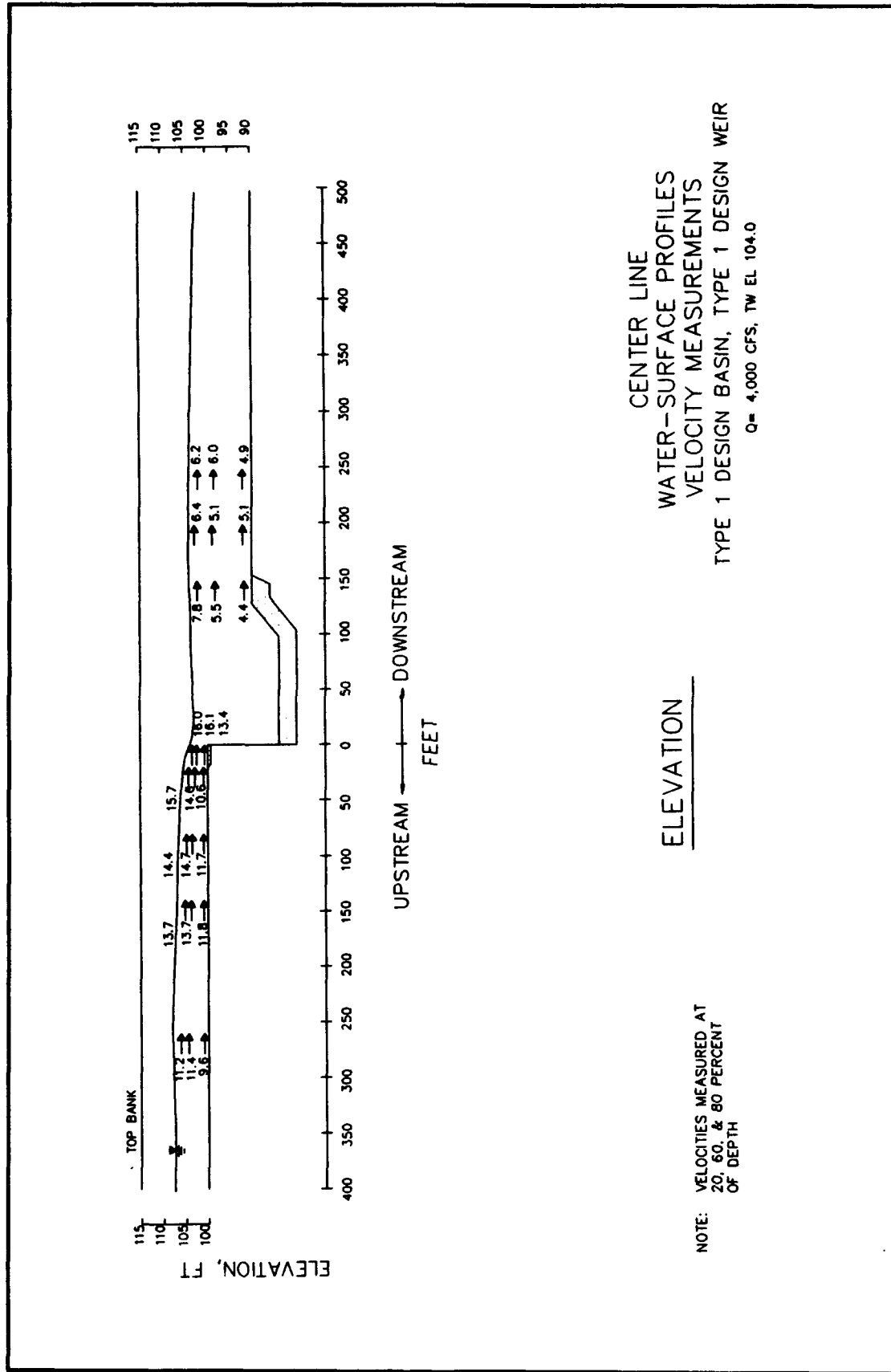
Figure E21. Water-surface profile and velocity measurements, Type 1 weir, Q = 4000 cfs, TW el = 104.0
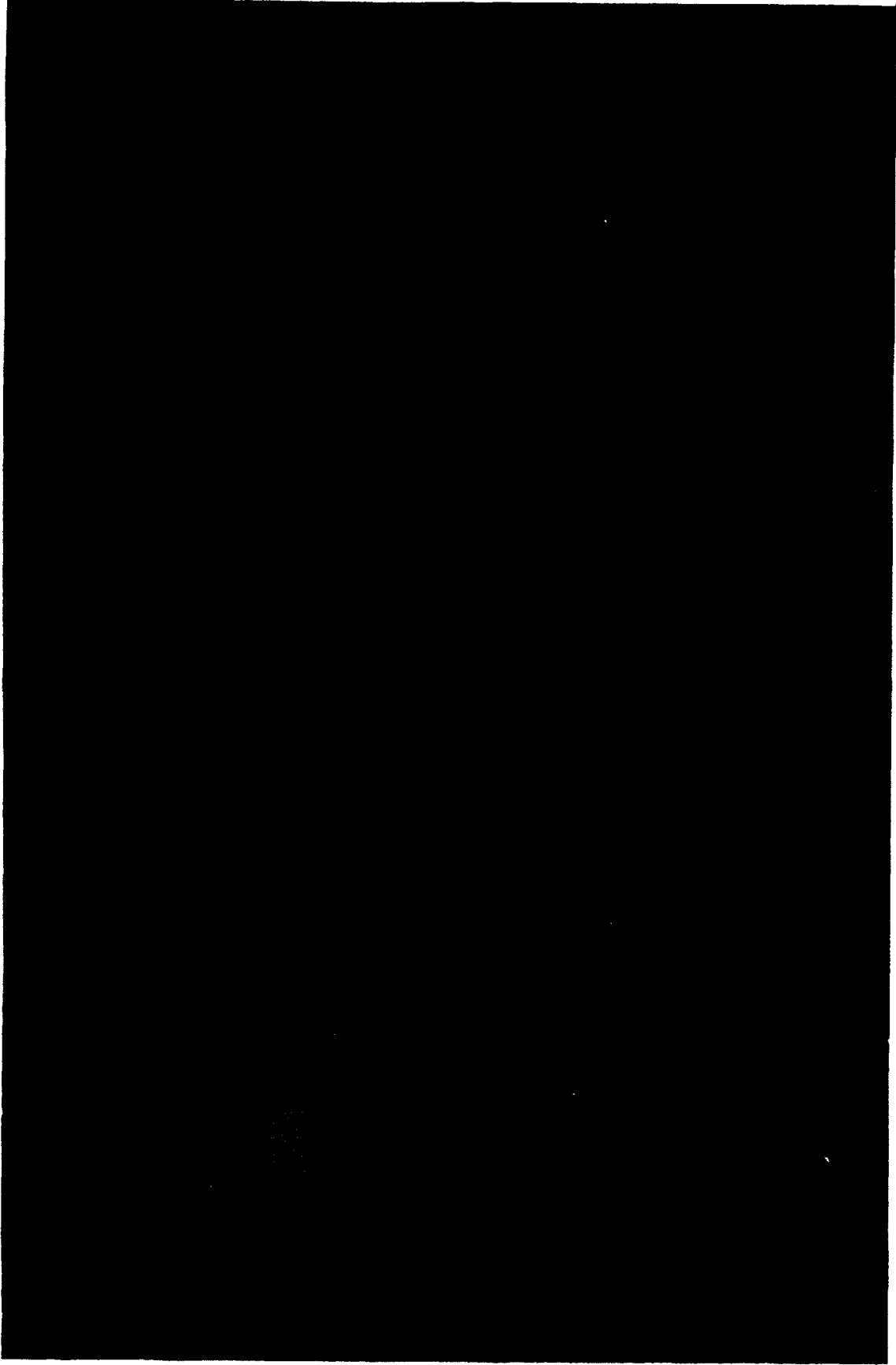
Figure E22. Type 1 weir, Q = 4000 cfs, TW el = 104.0
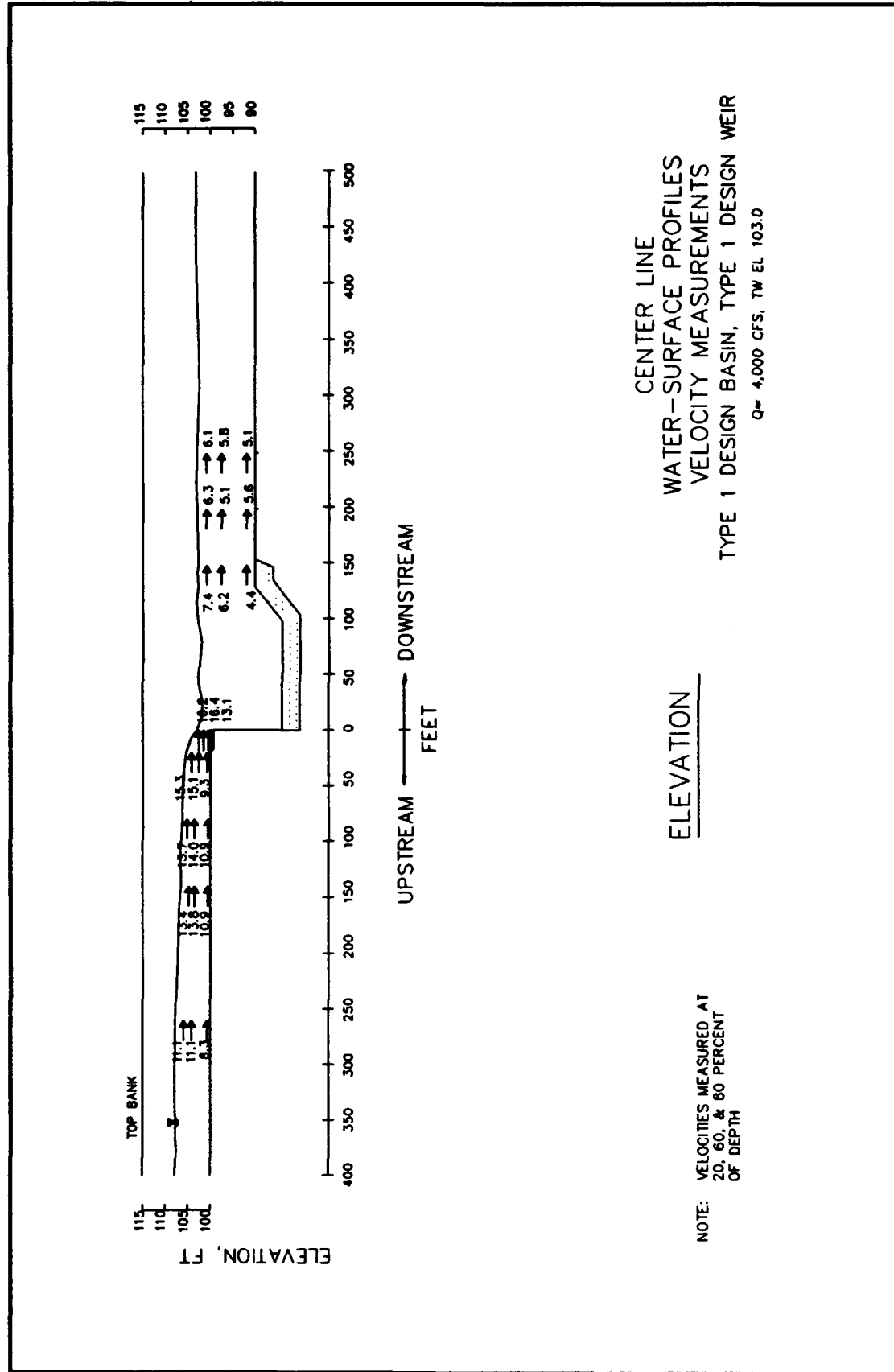
Figure E23. Water-surface profile and velocity measurements, Type 1 weir, Q = 4000 cfs, TW el = 103.0

Figure E24. Type 1 weir, Q = 4000 cfs, TW el = 103.0
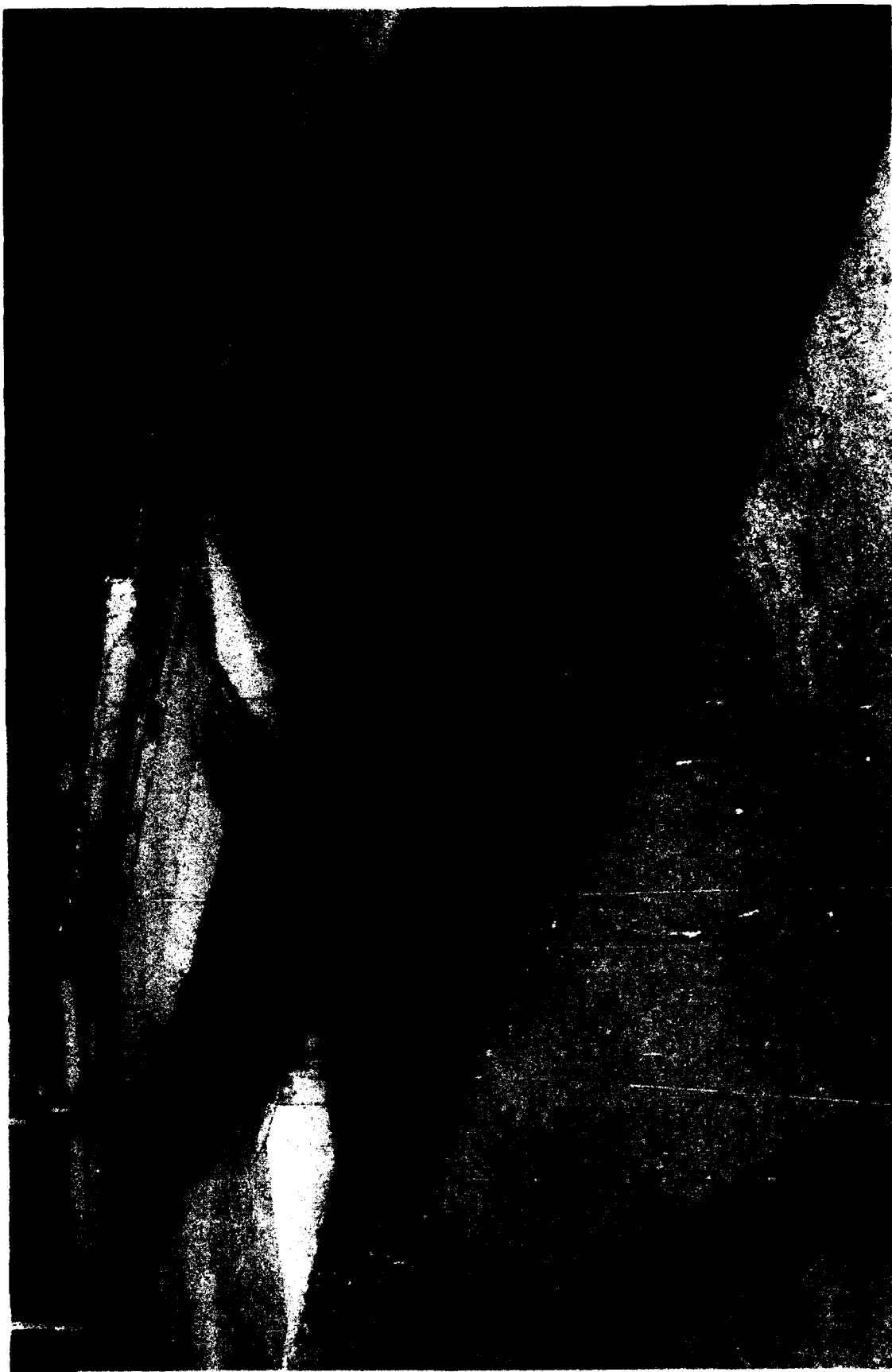
Figure E25. Type 1 weir, Q = 4000 cfs, TW el = 102.0
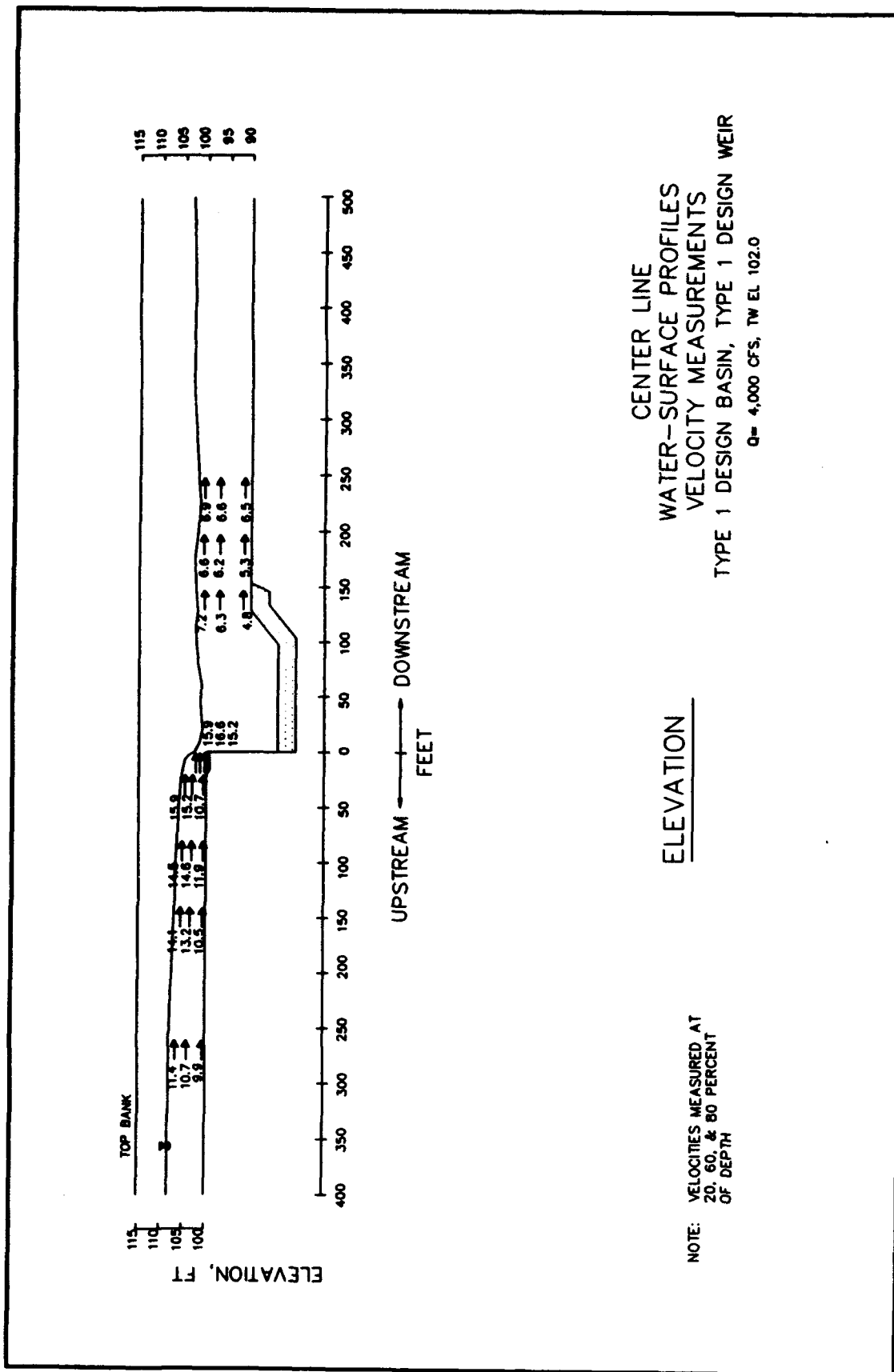
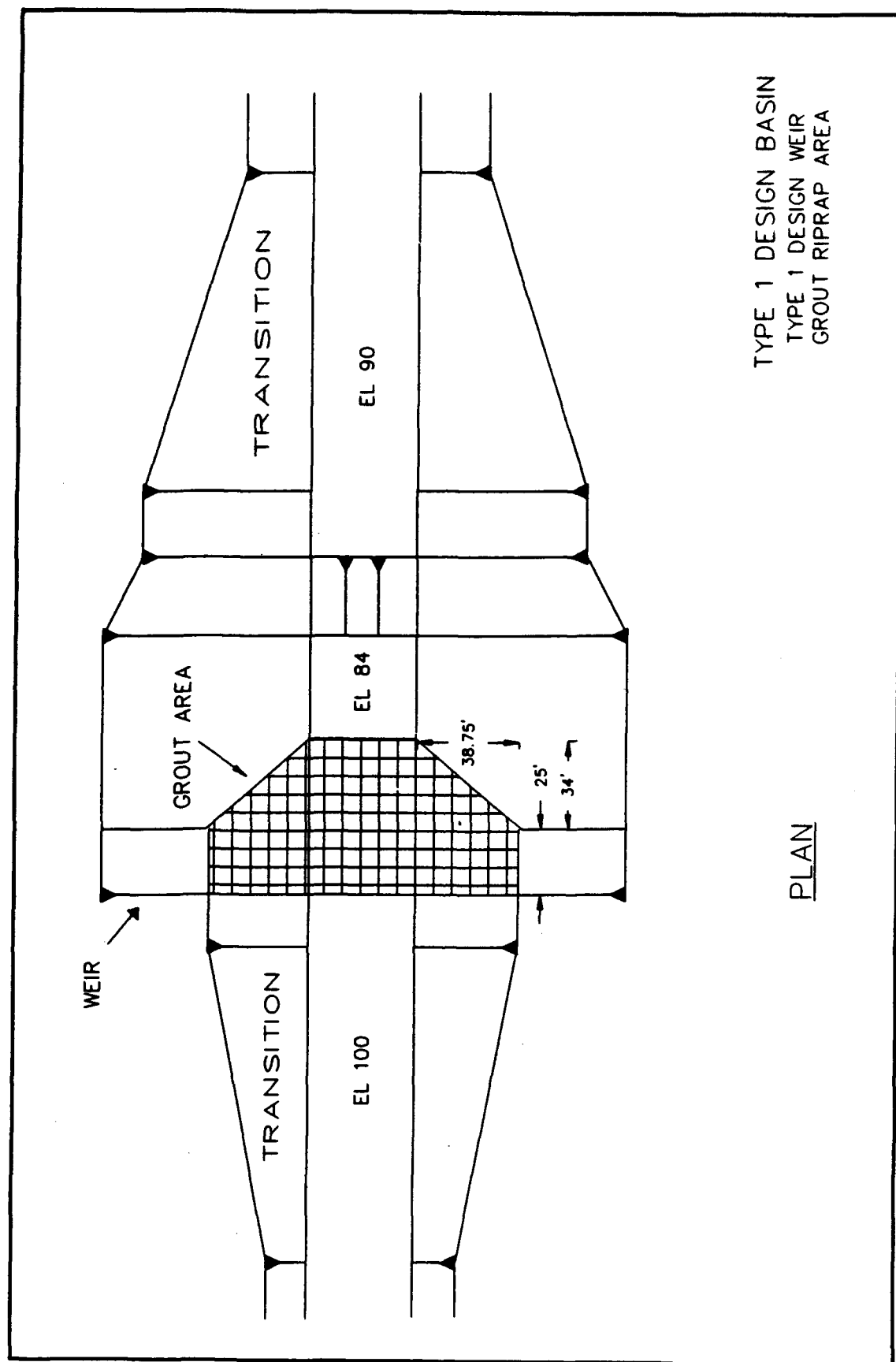Figure E26. Water-surface profile and velocity measurements, Type 1 weir, Q = 4000 cfs, TW el = 102.0
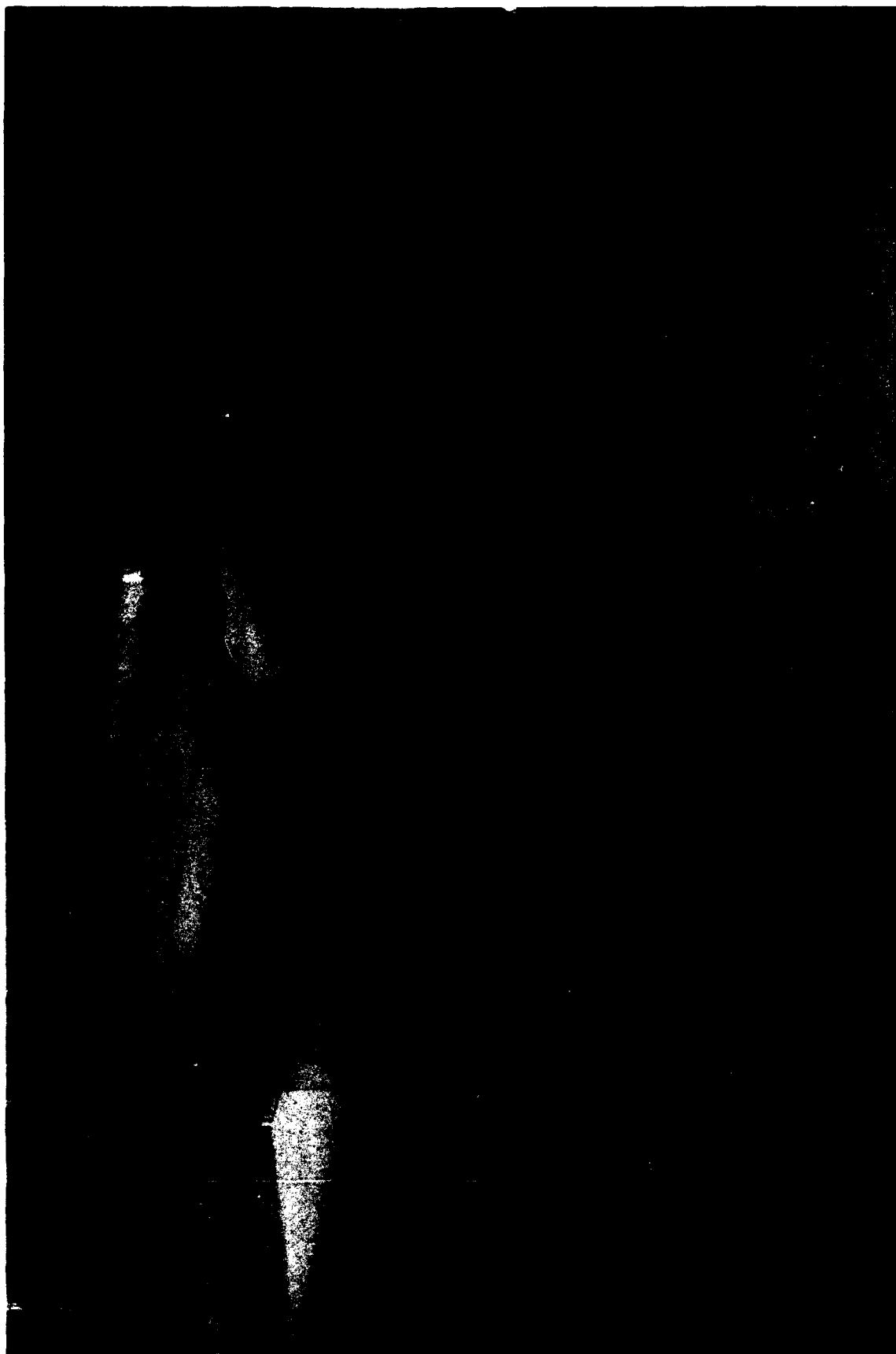
Figure E27. Type 1 weir, Q = 4000 cfs, TW el = 101.0

TYPE 1 DESIGN BASIN
TYPE 1 DESIGN WEIR
SCOUR AREAS
Q= 4,000 CFS, TW EL 101

PLAN
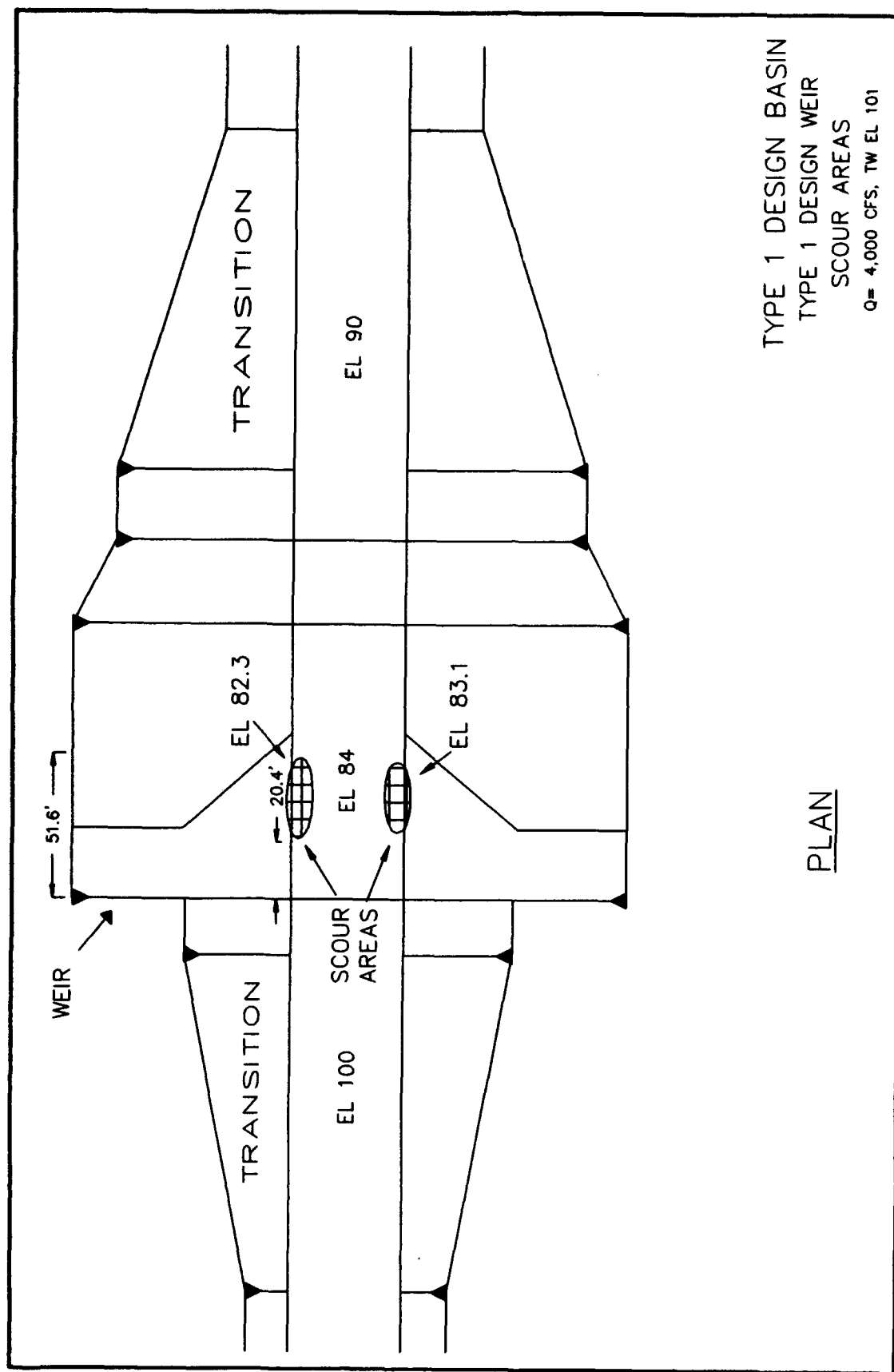
Figure E28. Scour areas, R1500 stone, Q = 4000 cfs, TW el = 101.0
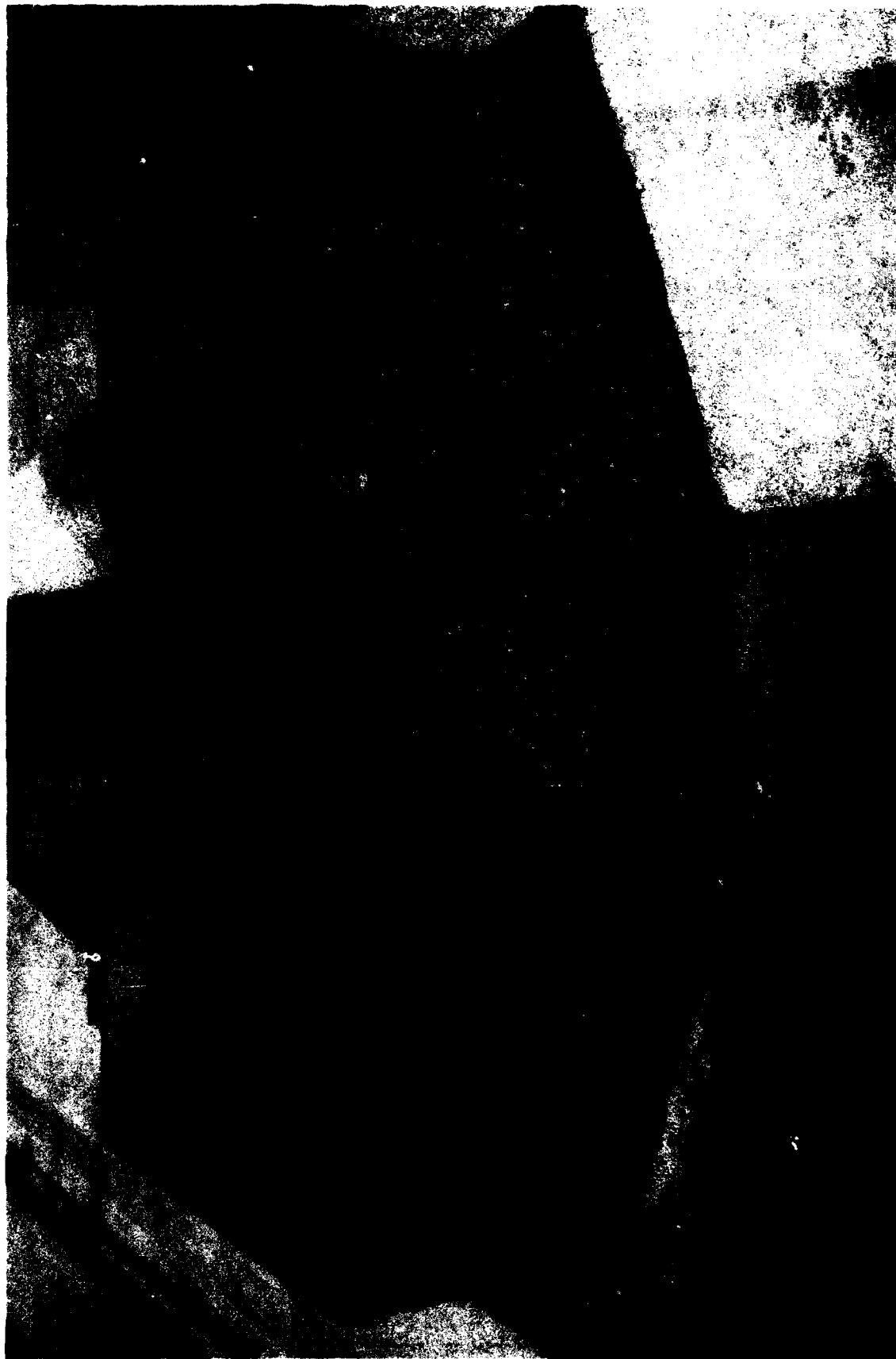
Figure E29. Failed riprap, R1500 stone, Q = 4000 cfs, TW el = 101.0
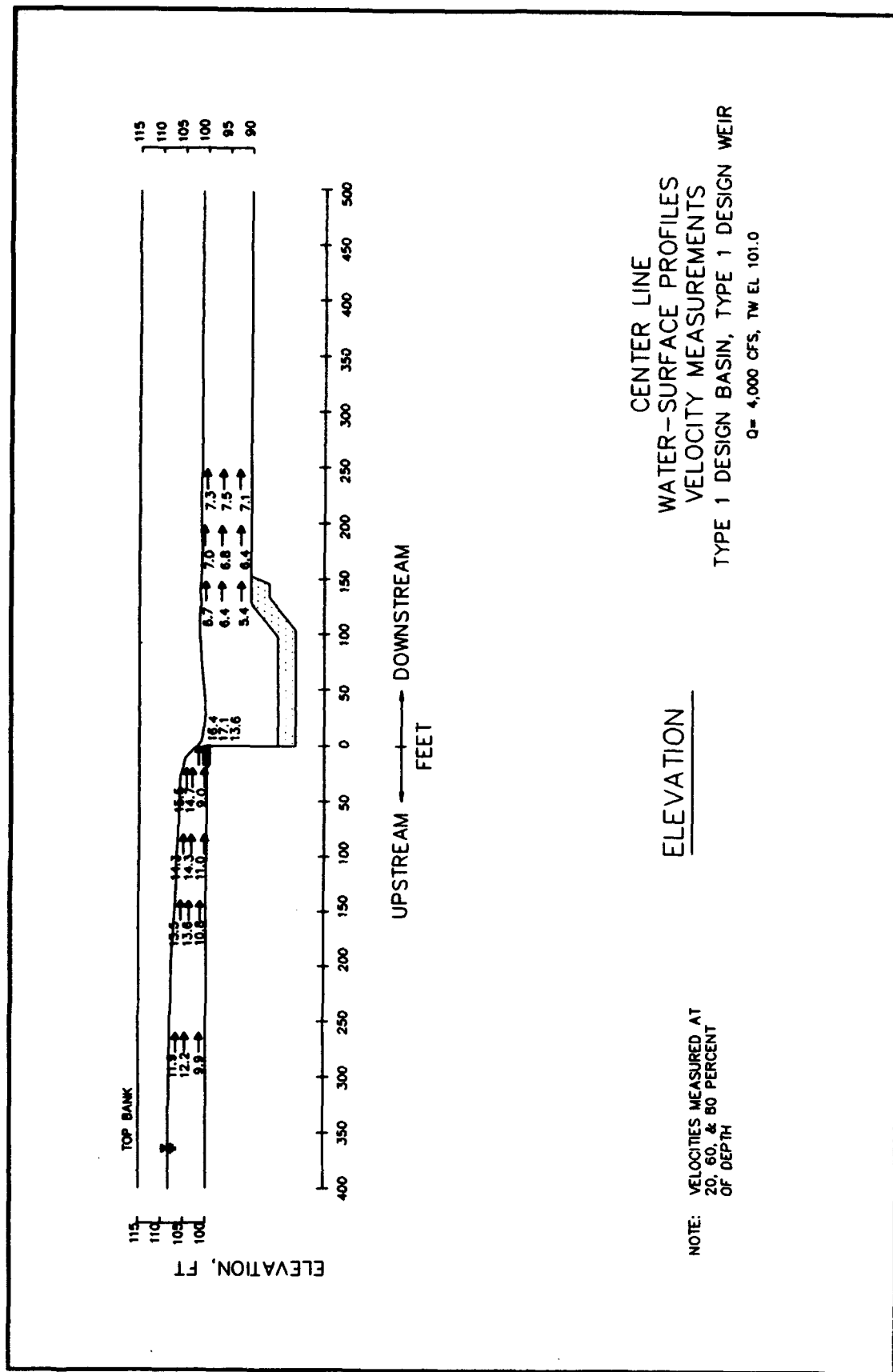
Figure E30. Water-surface profile and velocity measurements, Q = 4000 cfs, TW el = 101.0
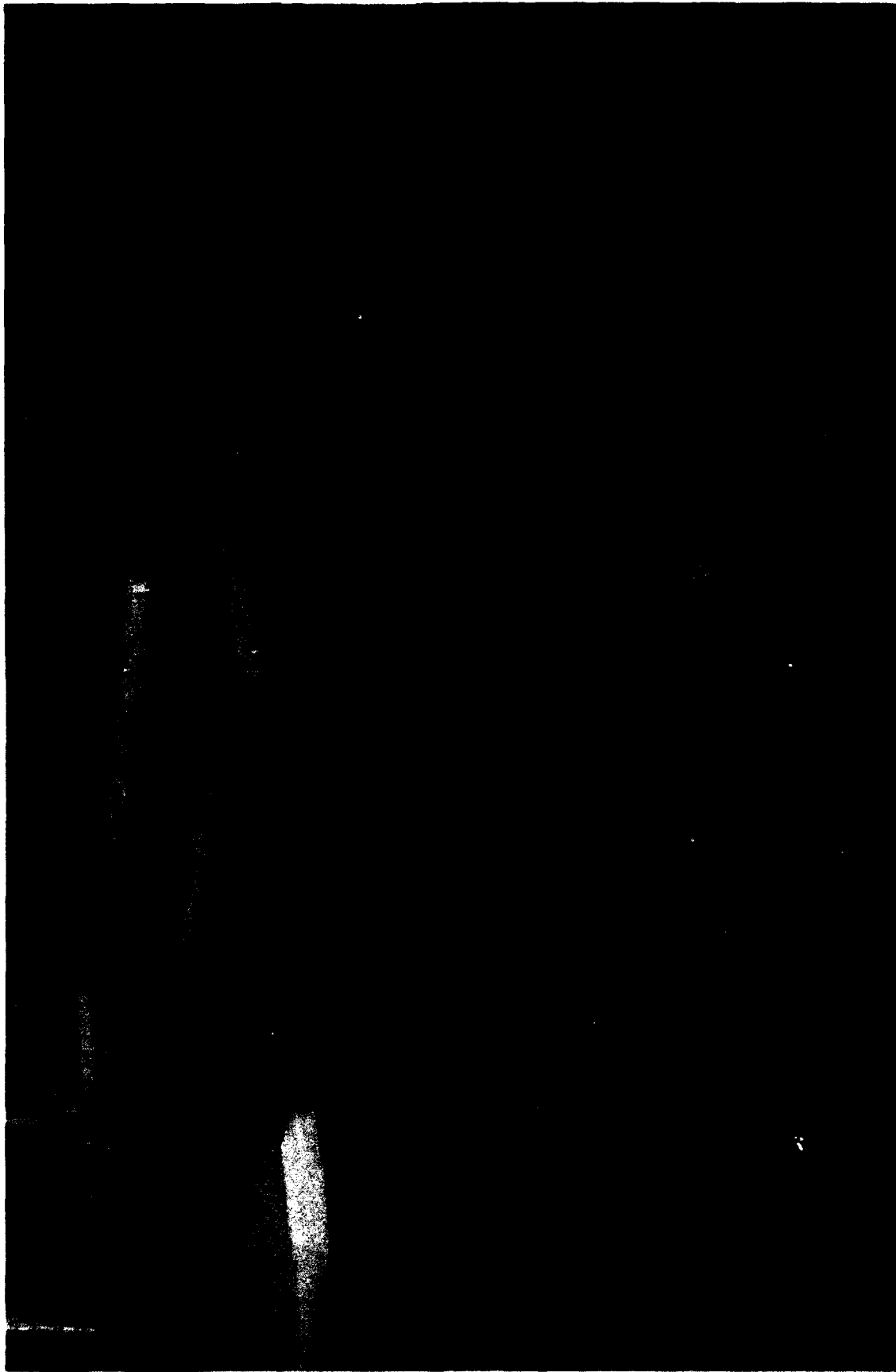
Figure E31. Type 2 weir, Q = 4000 cfs, TW el = 109.0

CENTER LINE
WATER–SURFACE PROFILES
VELOCITY MEASUREMENTS
TYPE 1 DESIGN BASIN, TYPE 2 DESIGN WEIR

Q = 4,000 CFS, TW EL 109.0

ELEVATION

NOTE: VELOCITIES MEASURED AT
        20, 60, & 80 PERCENT
        OF DEPTH
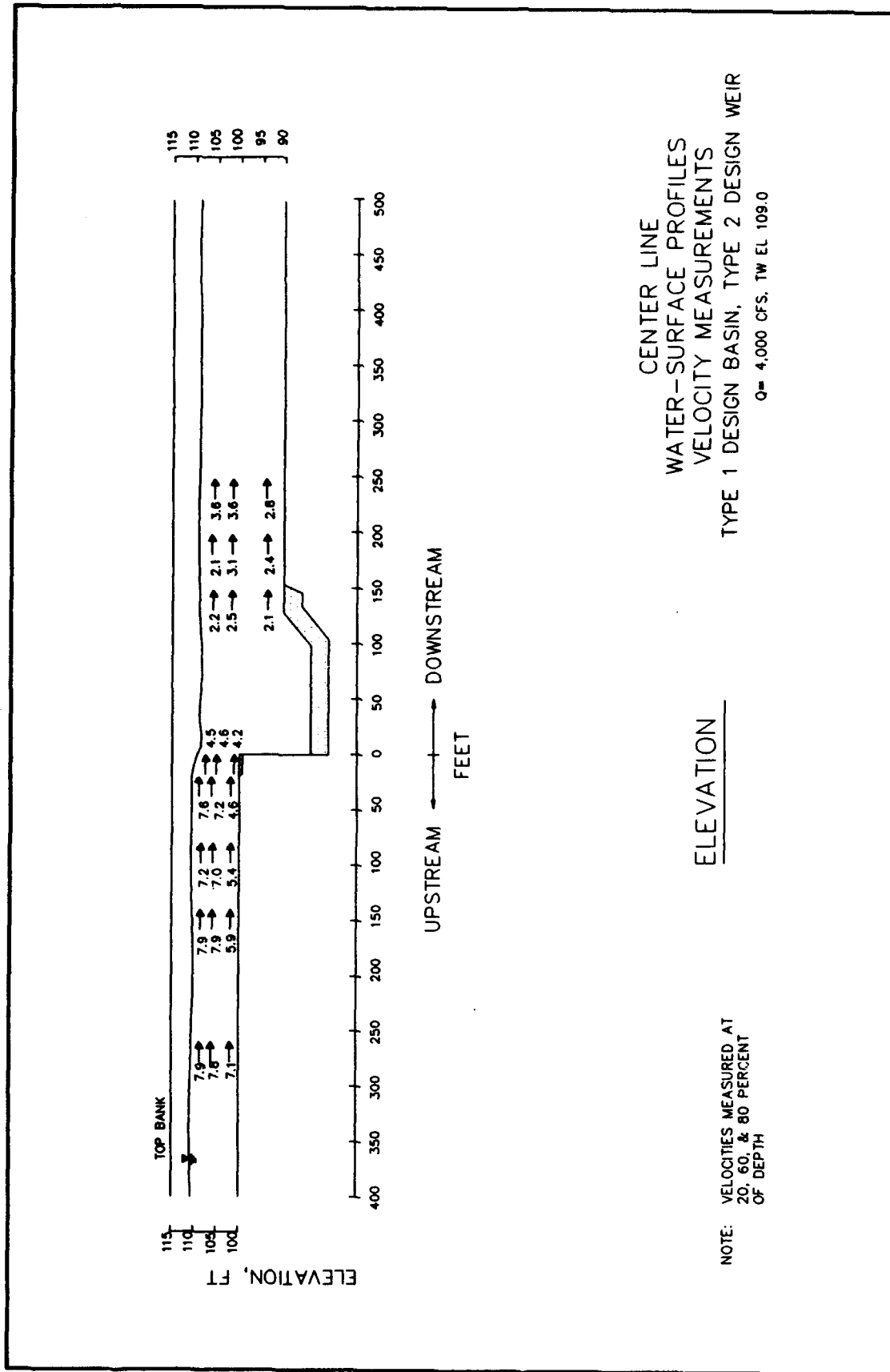
Figure E32. Water-surface profile and velocity measurements, Type 2 weir, Q = 4000 cfs, TW el = 109.0

TOP BANK

115 —
110 —
105 —
100 —
95 —
90 —

8.6 →   8.7 →   8.1 →   8.8 →
8.3 →   8.9 →   8.0 →   8.1 →
7.3 →   6.6 →   5.5 →   5.3 →

5.9 →
6.3 →
5.9 →

2.9 →   3.1 →   3.6 →
2.2 →   3.2 →   4.2 →

4.2 →   2.4 →   4.3 →

ELEVATION, FT
115
110
105
100

400   350   300   250   200   150   100   50   0   50   100   150   200   250   300   350   400   450   500

UPSTREAM ⟶   ⟵ DOWNSTREAM

FEET

NOTE: VELOCITIES MEASURED AT
20, 60, & 80 PERCENT
OF DEPTH

ELEVATION

CENTER LINE
WATER–SURFACE PROFILES
VELOCITY MEASUREMENTS
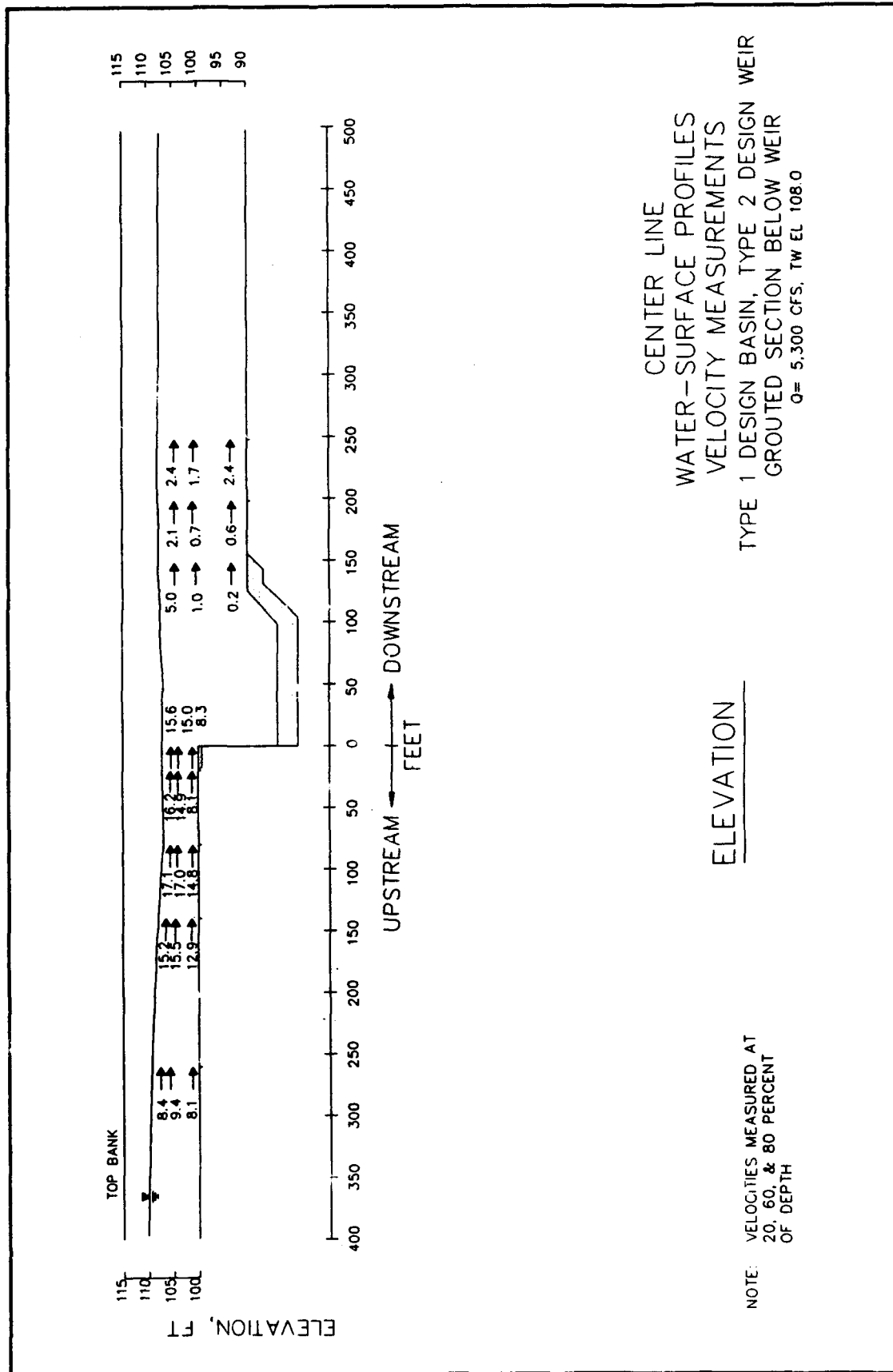TYPE 1 DESIGN BASIN, TYPE 2 DESIGN WEIR

Q= 4,000 CFS, TW EL 108.0

Figure E33. Water-surface profile and velocity measurements, Type 2 weir, Q = 4000 cfs, TW el = 108.0

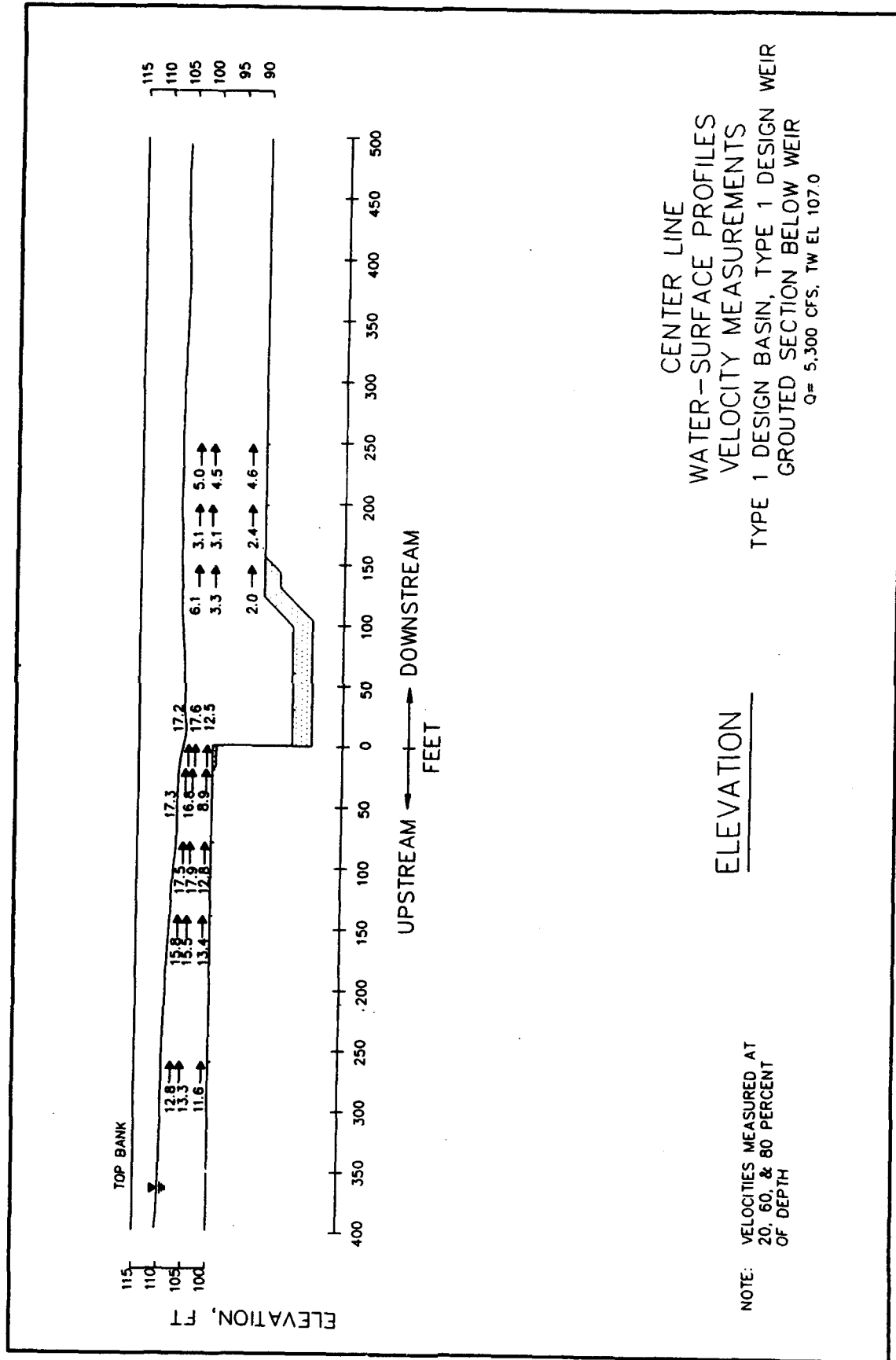Figure E34. Water-surface profile and velocity measurements, Type 2 weir, Q = 4000 cfs, TW el = 107.0

Figure E35. Type 2 weir, Q = 4000 cfs, TW el = 105.0

Figure E36. Water-surface profile and velocity measurements, Type 2 weir, Q = 4000 cfs, TW el = 105.0

CENTER LINE
WATER-SURFACE PROFILES
VELOCITY MEASUREMENTS
TYPE 1 DESIGN BASIN, TYPE 2 DESIGN WEIR

Q= 4,000 CFS, TW EL 104.0

ELEVATION

NOTE: VELOCITIES MEASURED AT
20, 60, & 80 PERCENT
OF DEPTH

Figure E37. Water-surface profile and velocity measurements, Type 2 weir, Q = 4000 cfs, TW el = 104.0

Figure E38. Scour area, R1500 stone, Type 2 weir, Q = 4000 cfs, TW el = 103.0

Figure E39. Failed riprap, R1500 stone, Type 2 weir, Q = 4000 cfs, TW el = 103.0
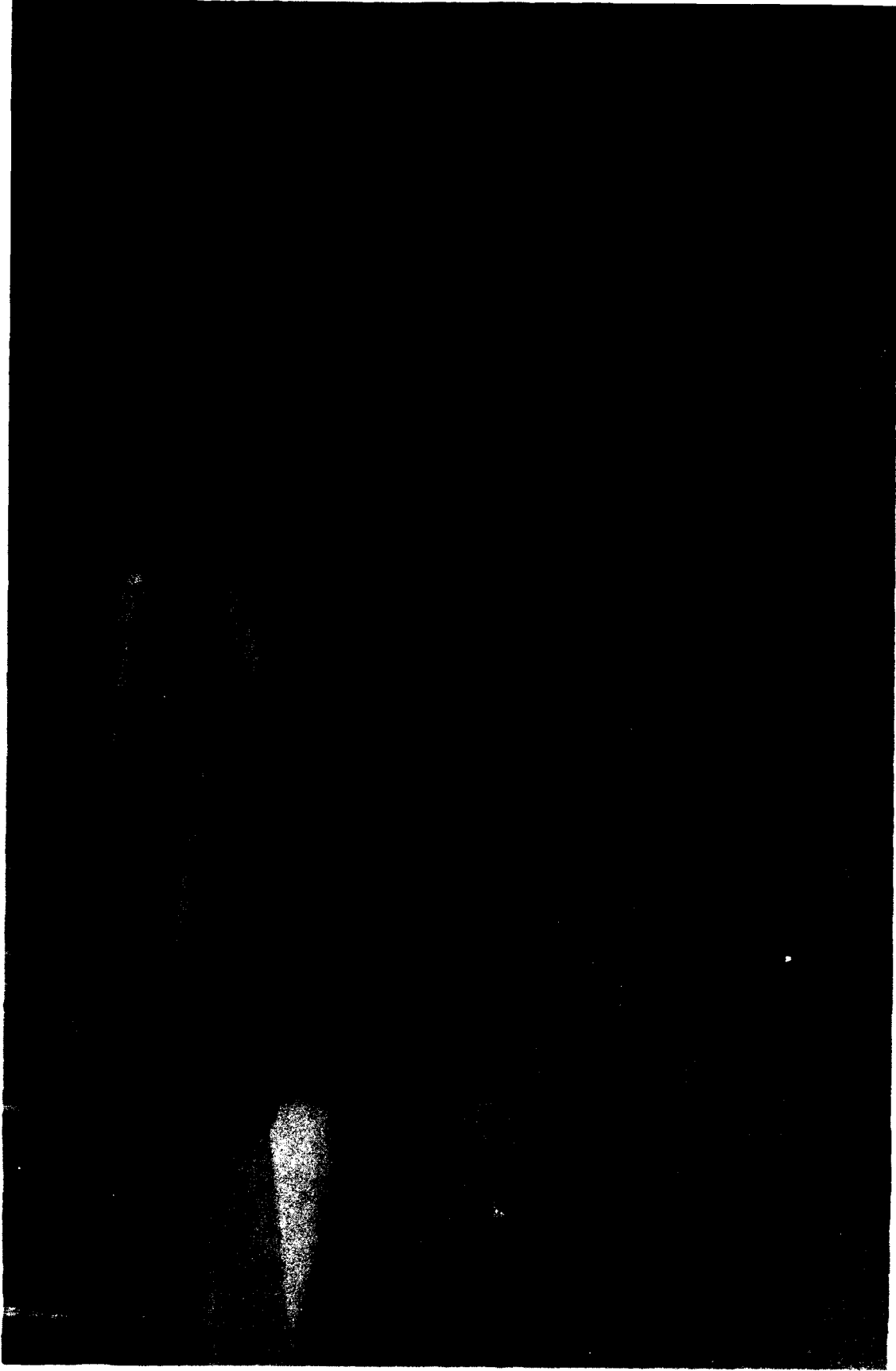
Figure E40. Type 2 weir, Q = 4000 cfs, TW el = 103.0

CENTER LINE
WATER—SURFACE PROFILES
VELOCITY MEASUREMENTS
TYPE 1 DESIGN BASIN, TYPE 2 DESIGN WEIR

Q= 4,000 CFS, TW EL 103.0

ELEVATION

NOTE: VELOCITIES MEASURED AT
20, 60, & 80 PERCENT
OF DEPTH
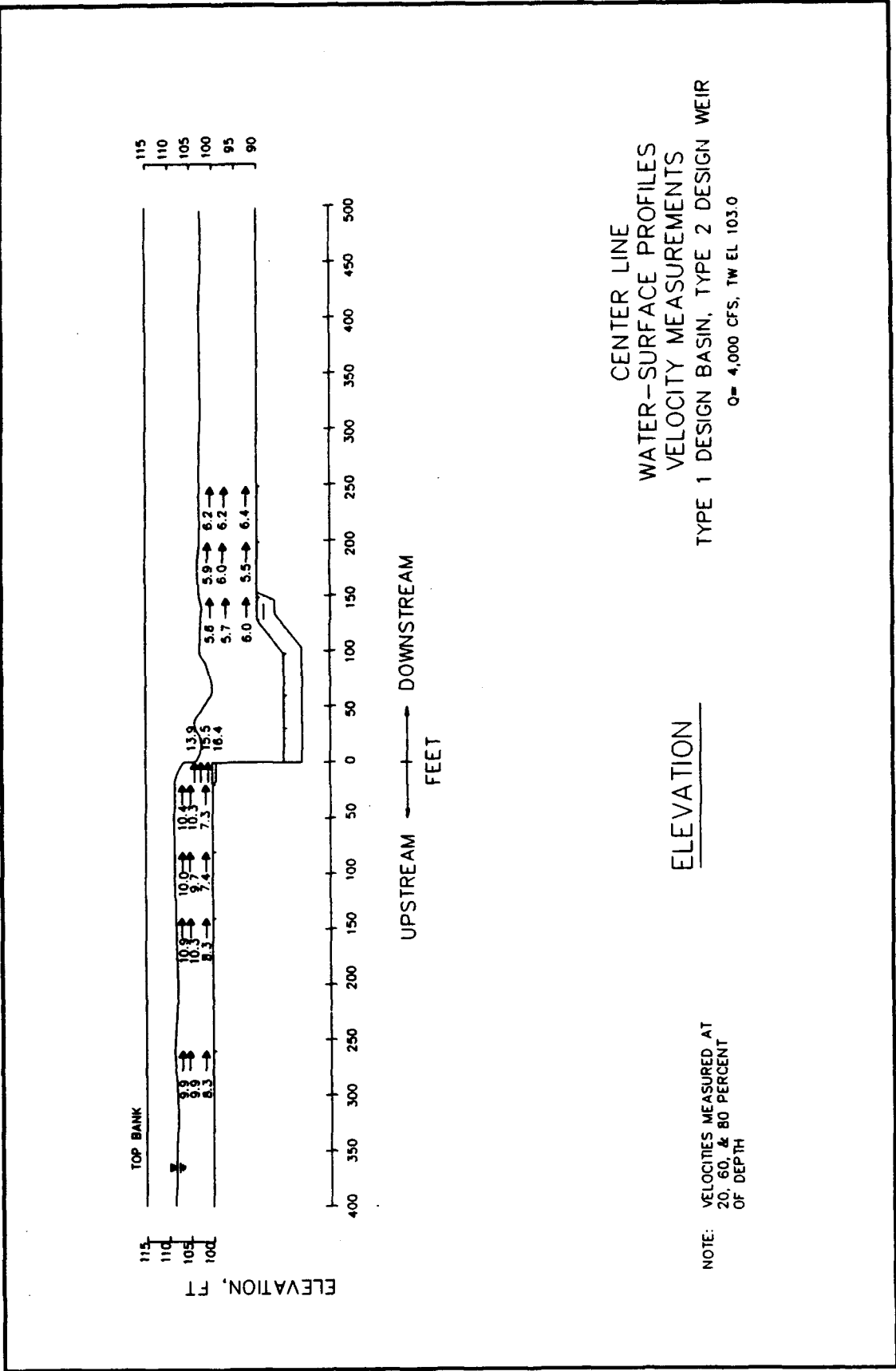
Figure E41. Water-surface profile, Type 2 weir, Q = 4000 cfs, TW el = 103.0

TYPE 1 DESIGN BASIN
TYPE 1 DESIGN WEIR
GROUT RIPRAP AREA

TRANSITION

EL 90

GROUT AREA

EL 84

38.75'

25'

34'

WEIR

TRANSITION

EL 100

PLAN

Figure E42. Plan view, Type 1 weir, grouted area in stilling basin

CENTER LINE
WATER-SURFACE PROFILES
VELOCITY MEASUREMENTS
TYPE 1 DESIGN BASIN, TYPE 1 DESIGN WEIR
GROUTED SECTION D/S WEIR
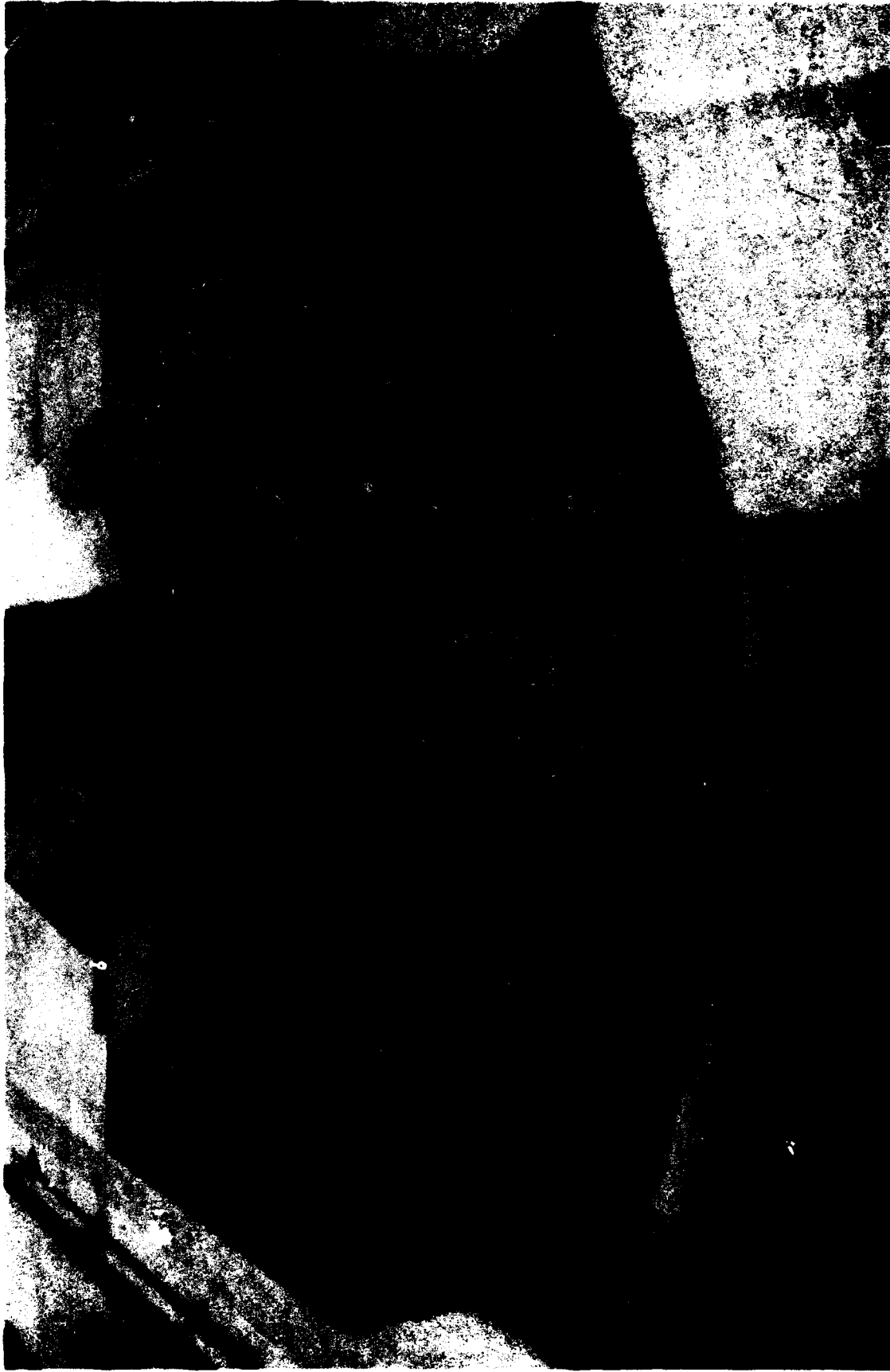Q= 4,000 CFS, TW EL 100.0
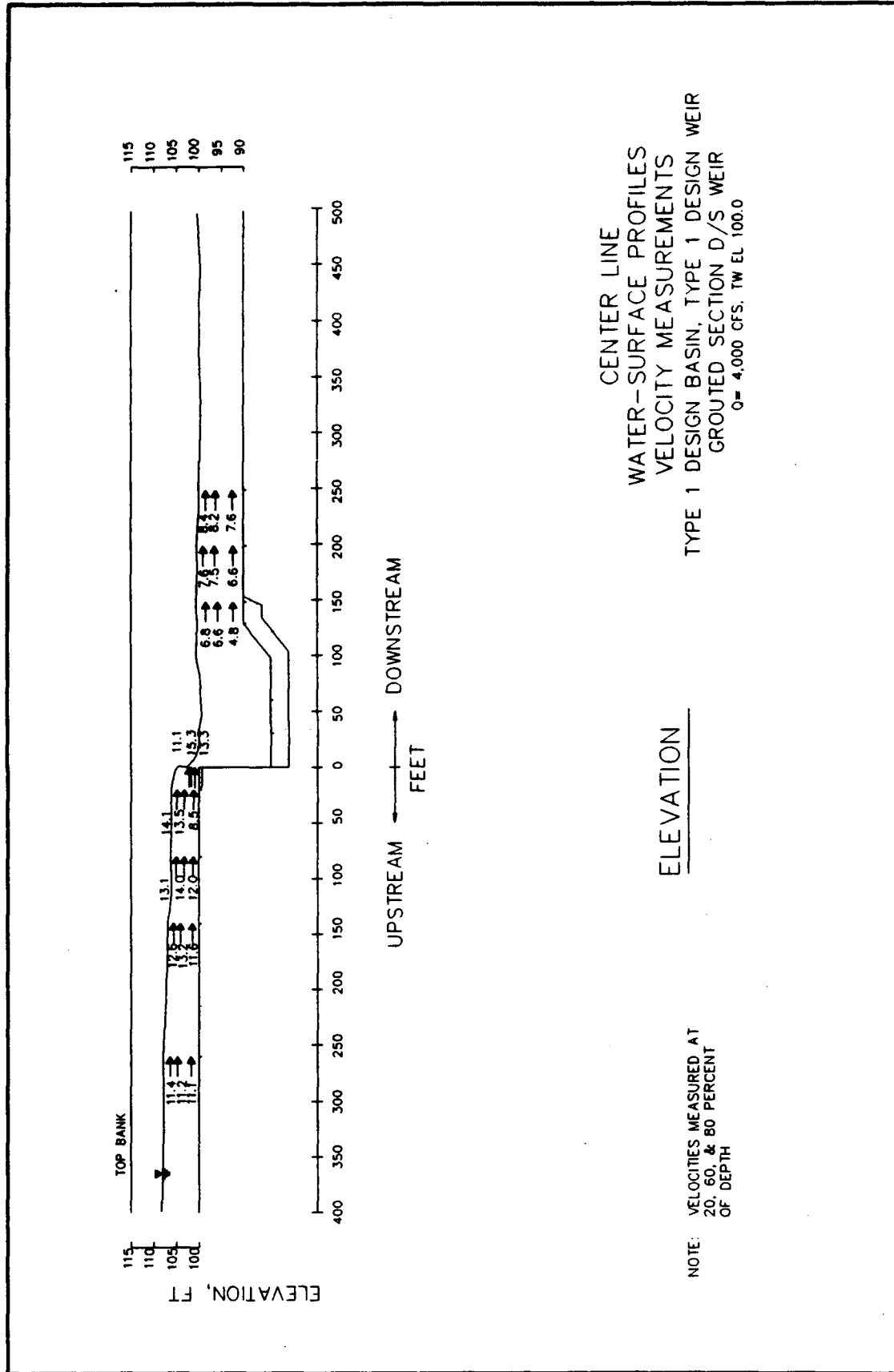
Figure E43. Water-surface profile and velocity measurements, grouted riprap, Q = 4000 cfs, TW el = 100.0
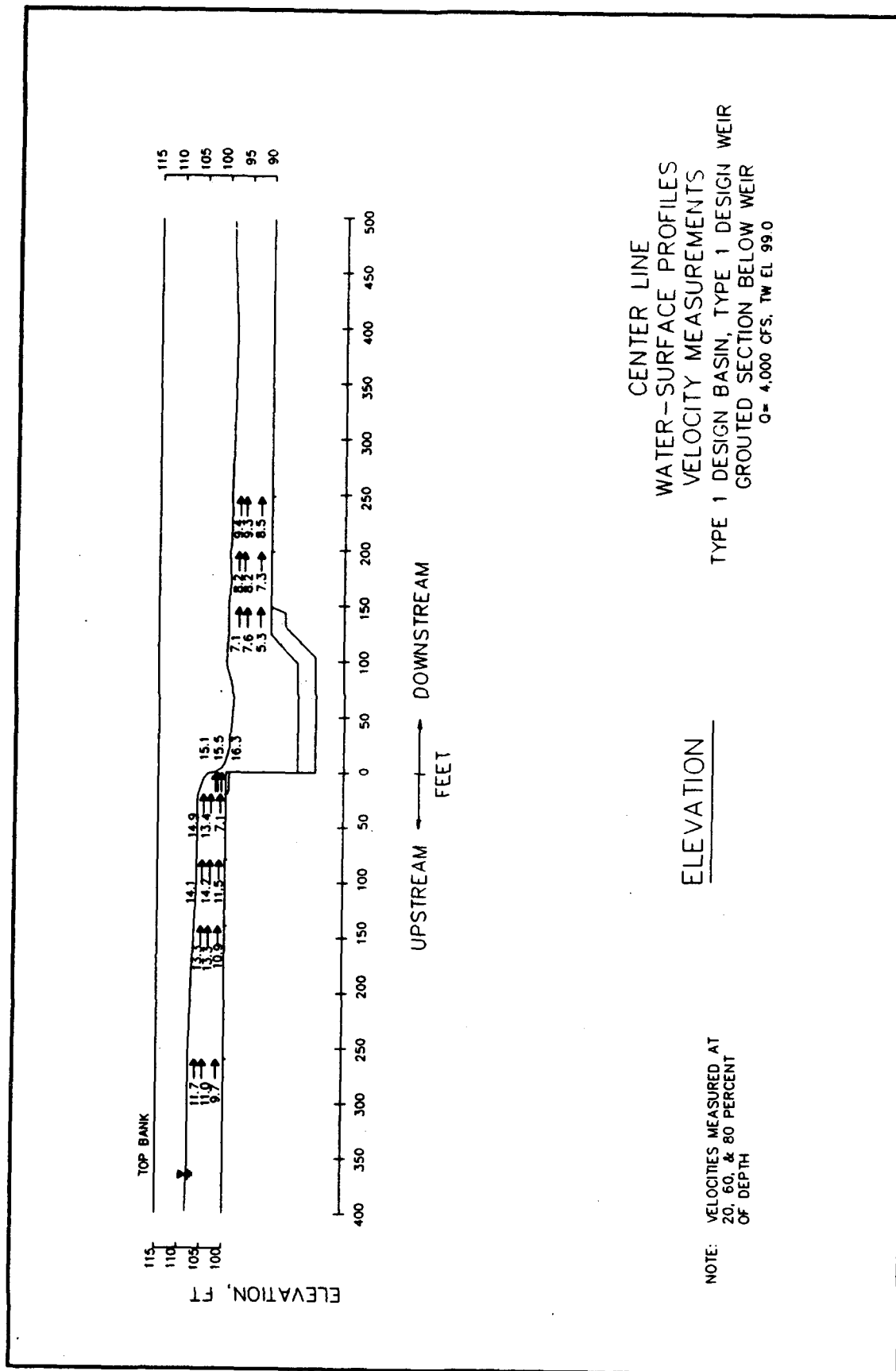
Figure E44. Water-surface profile and velocity measurments, grouted riprap, Q = 4000 cfs, TW el = 99.0

Figure E45. Type 1 weir, grouted riprap, Q = 5300 cfs, TW el = 109.0
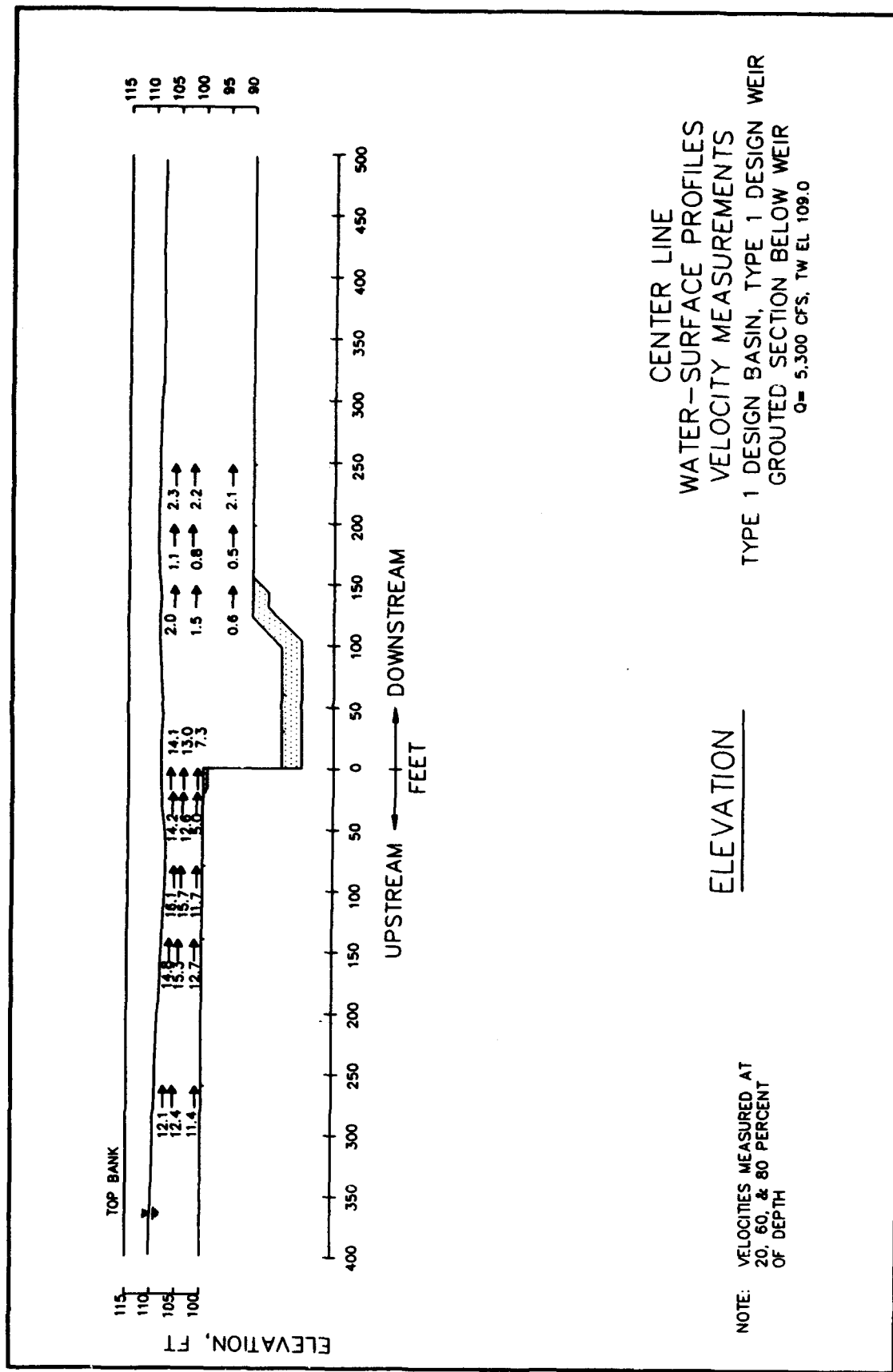
Figure E46. Water-surface profile and velocity measurements, grouted riprap, Q = 5300 cfs, TW el = 109.0

Figure E47. Water-surface profile and velocity measurements, grouted riprap, Q = 5300 cfs, TW el = 108.0

Figure E48. Water-surface profile and velocity measurements, grouted riprap, Q = 5300 cfs, TW el = 107.0
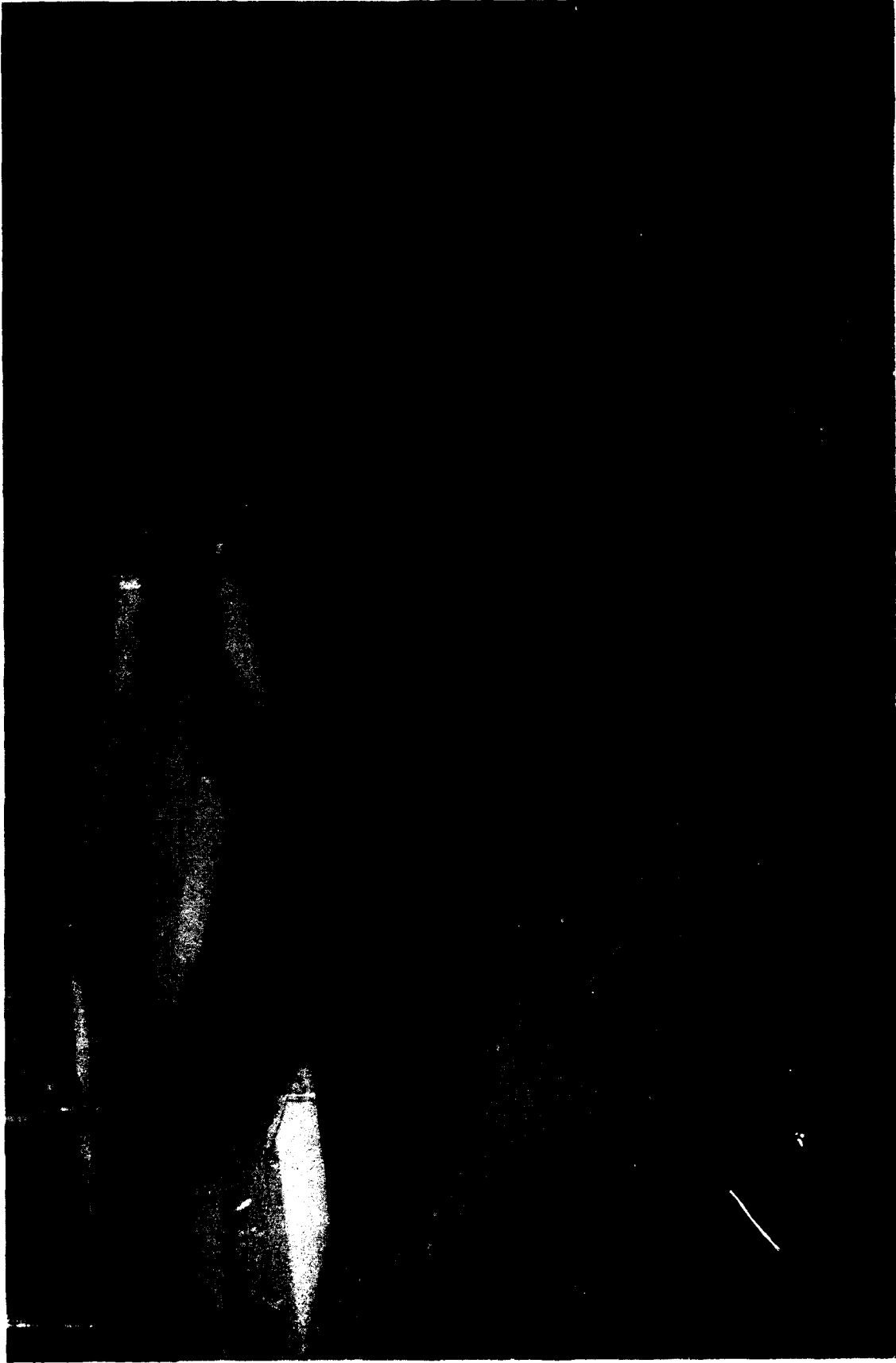
Figure E49. Type 1 weir, grouted riprap, Q = 5300 cfs, TW el = 107.0
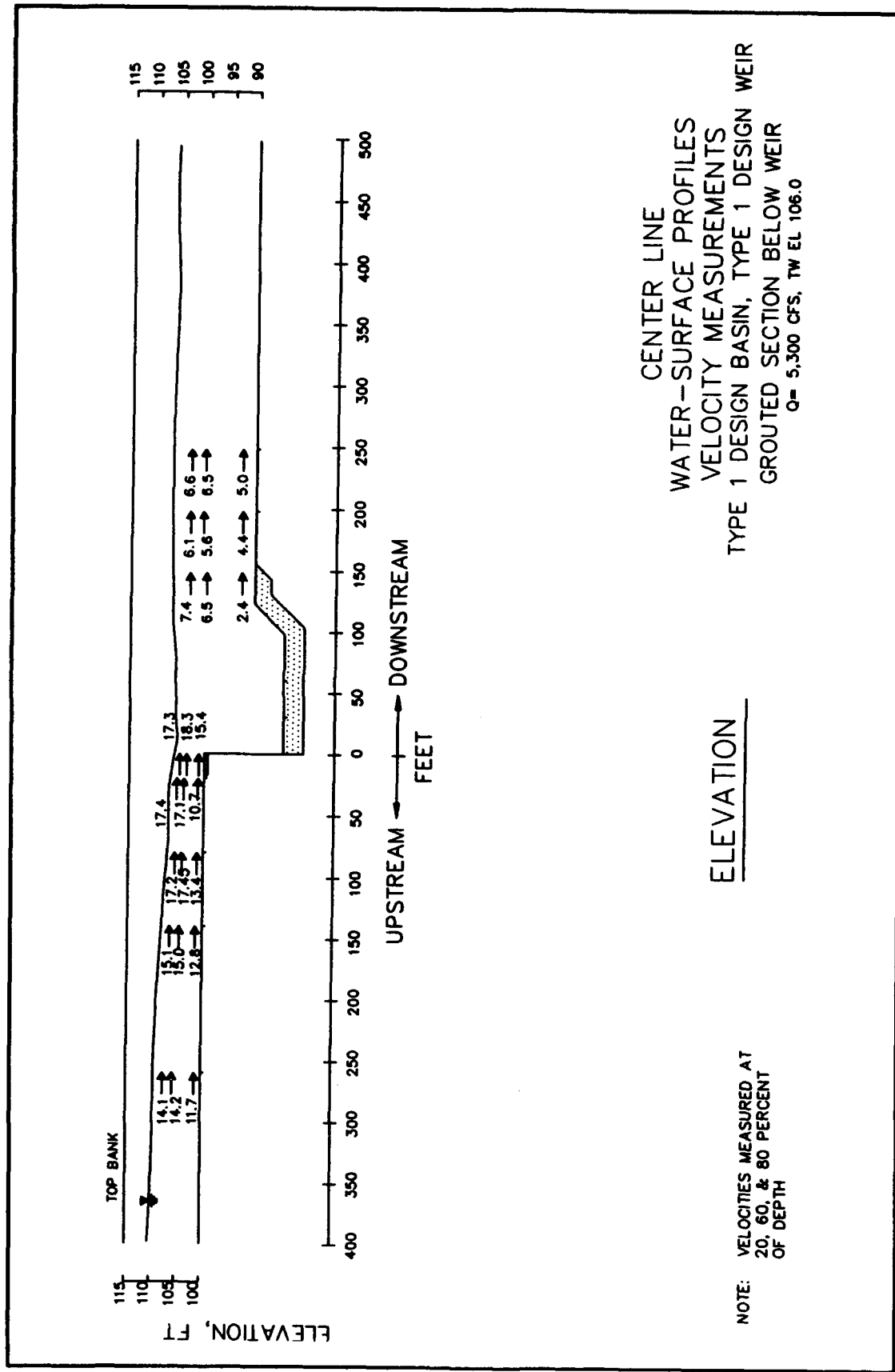
Figure E50. Water-surface profile and velocity measurements, grouted riprap, Q = 5300 cfs, TW el = 106.0
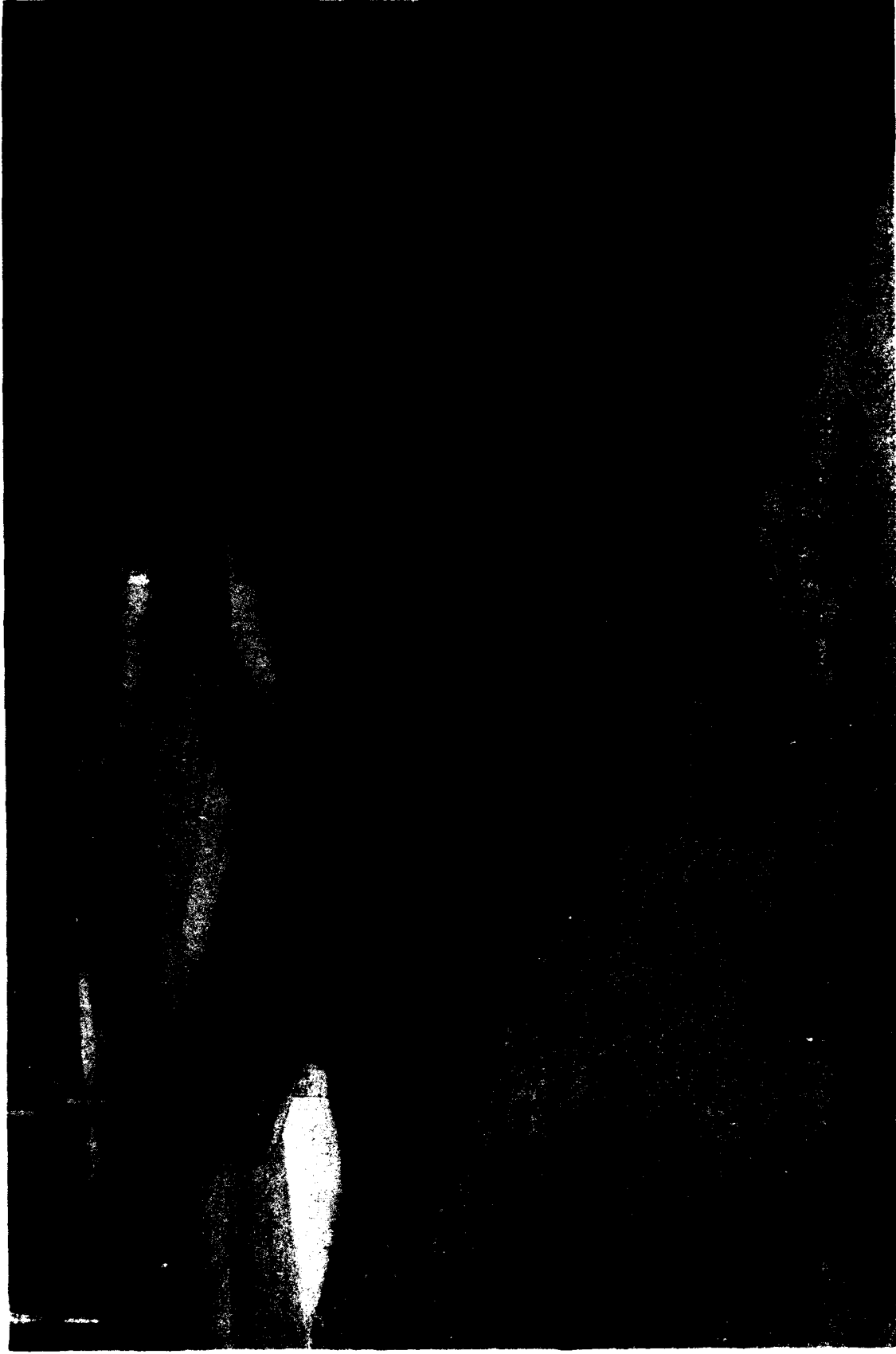
Figure E51. Type 1 weir, grouted riprap, Q = 5300 cfs, TW el = 105.0

PLAN

TYPE 1 DESIGN BASIN
TYPE 1 DESIGN WEIR
SCOUR AREAS
Q= 5,300 CFS, TW EL 105.0
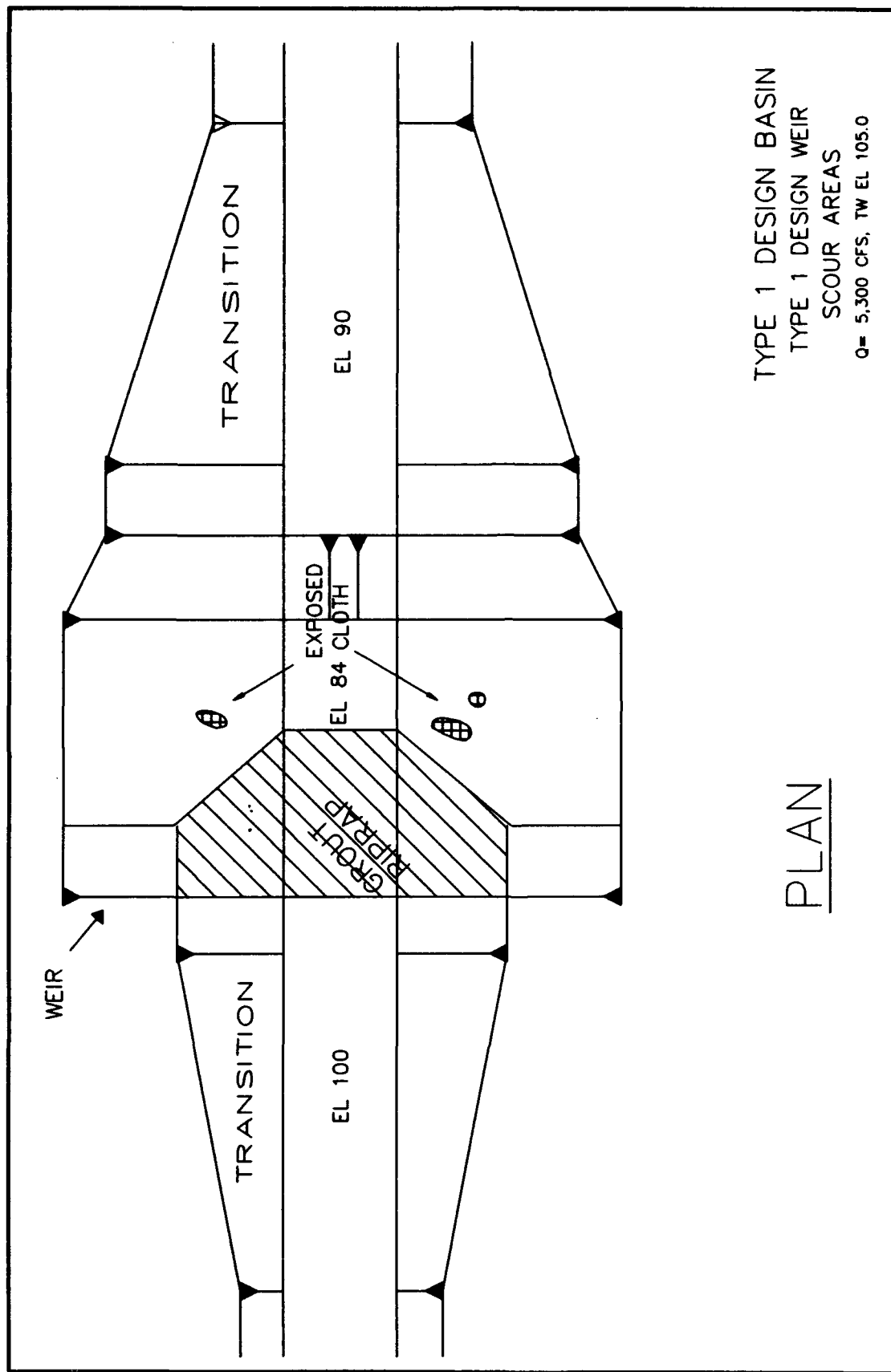
Figure E52. Scour areas, grouted basin, Q = 5300 cfs, TW el =105.0

Figure E53. Failed riprap, R200, Q = 5300 cfs, TW el = 105.0
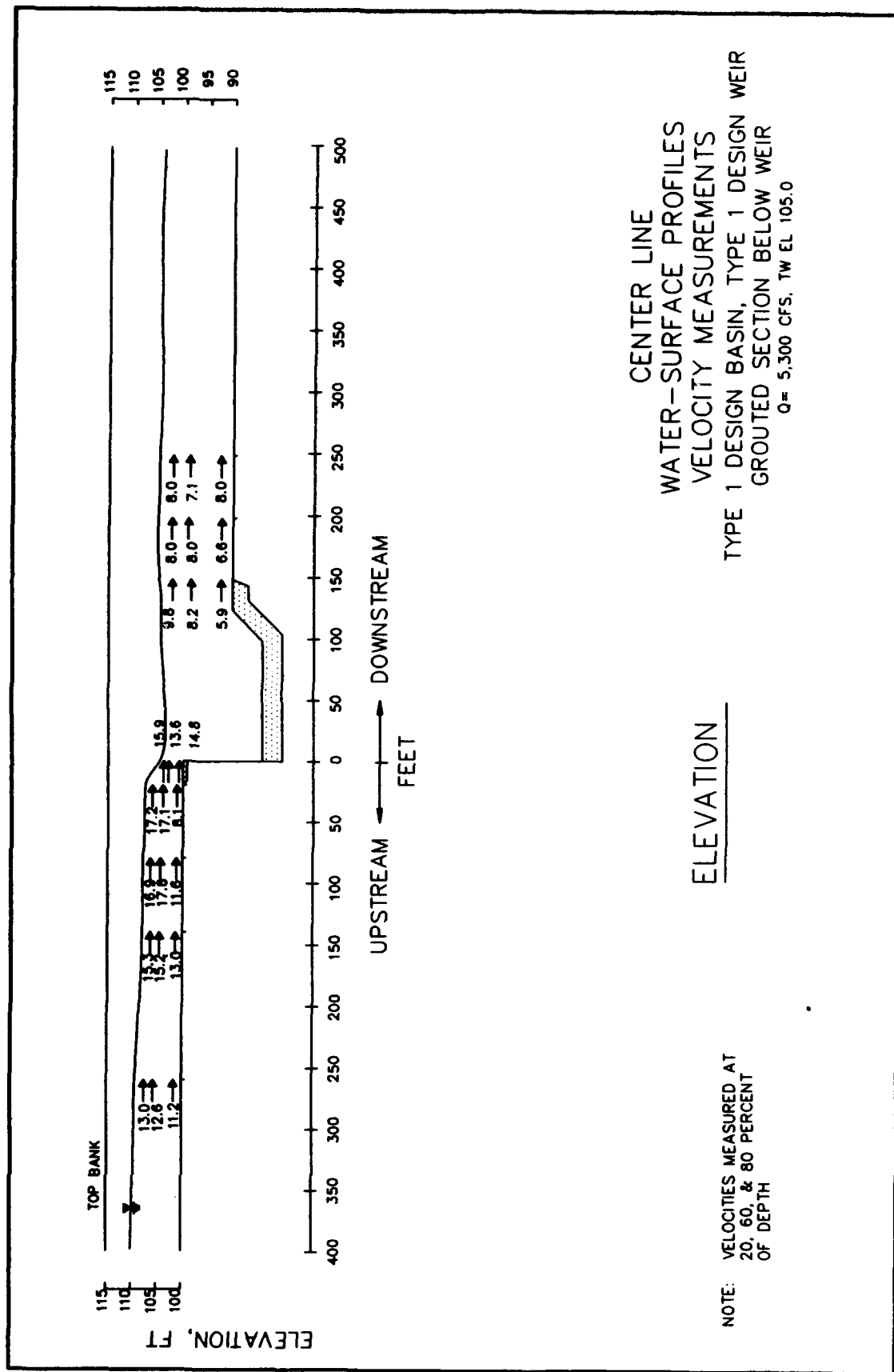
Figure E54. Water-surface profile and velocity measurements, grouted riprap, Q = 5300 cfs, TW el = 105.0
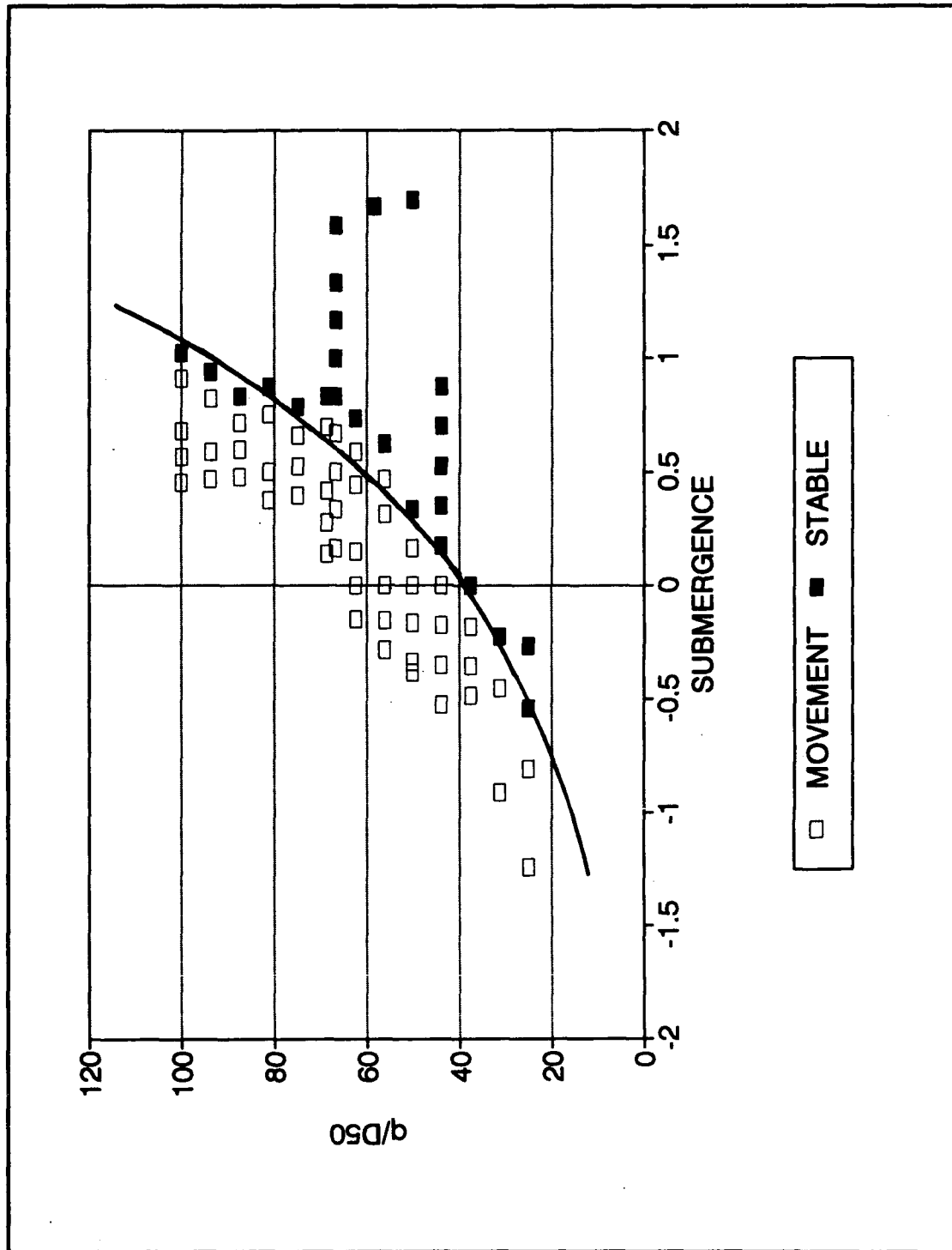
Figure E55. q/d50 versus submergence for large riprap sizes
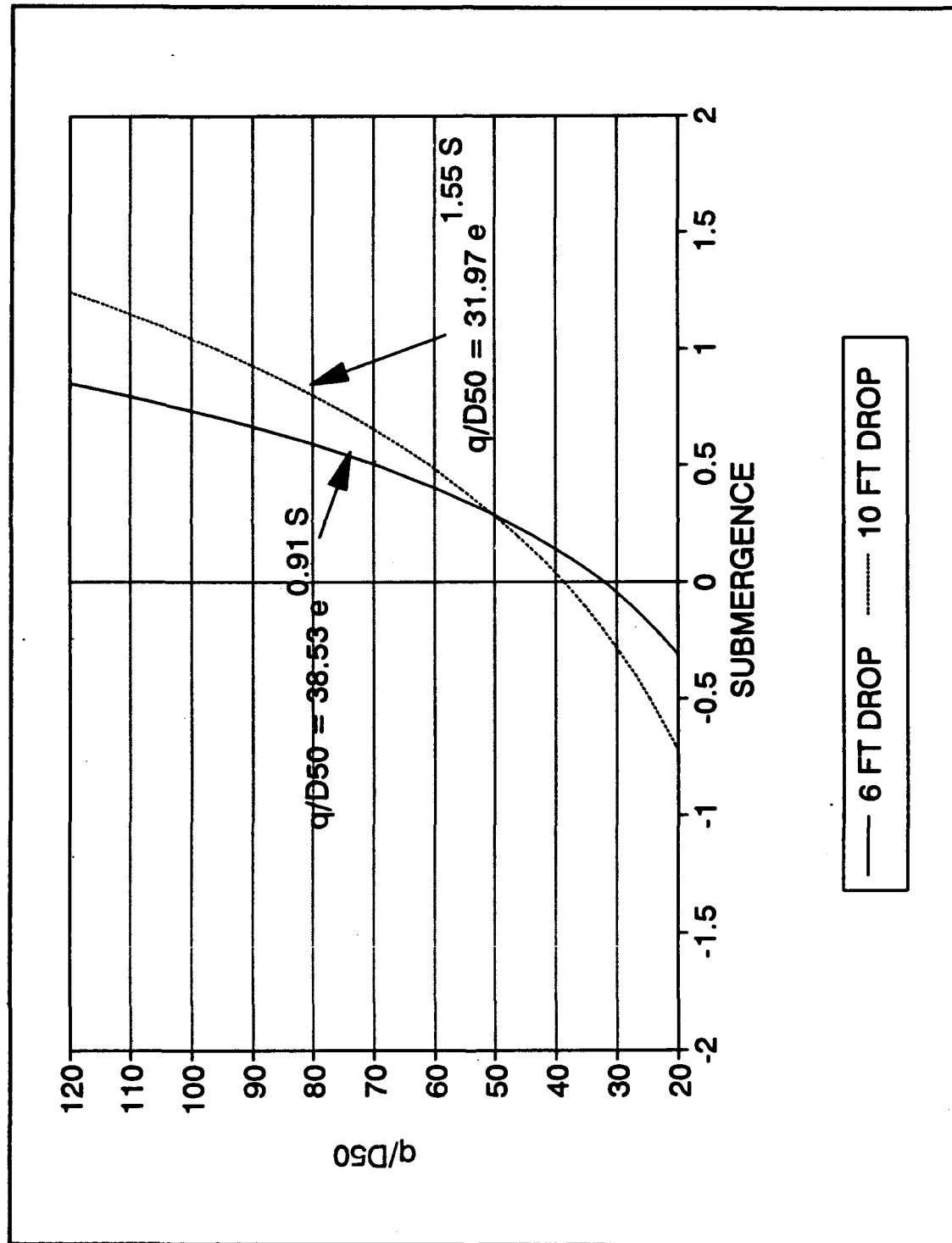
Figure E56. Comparison between modified equation for 10-ft drop and CSU equation for 6-ft drop
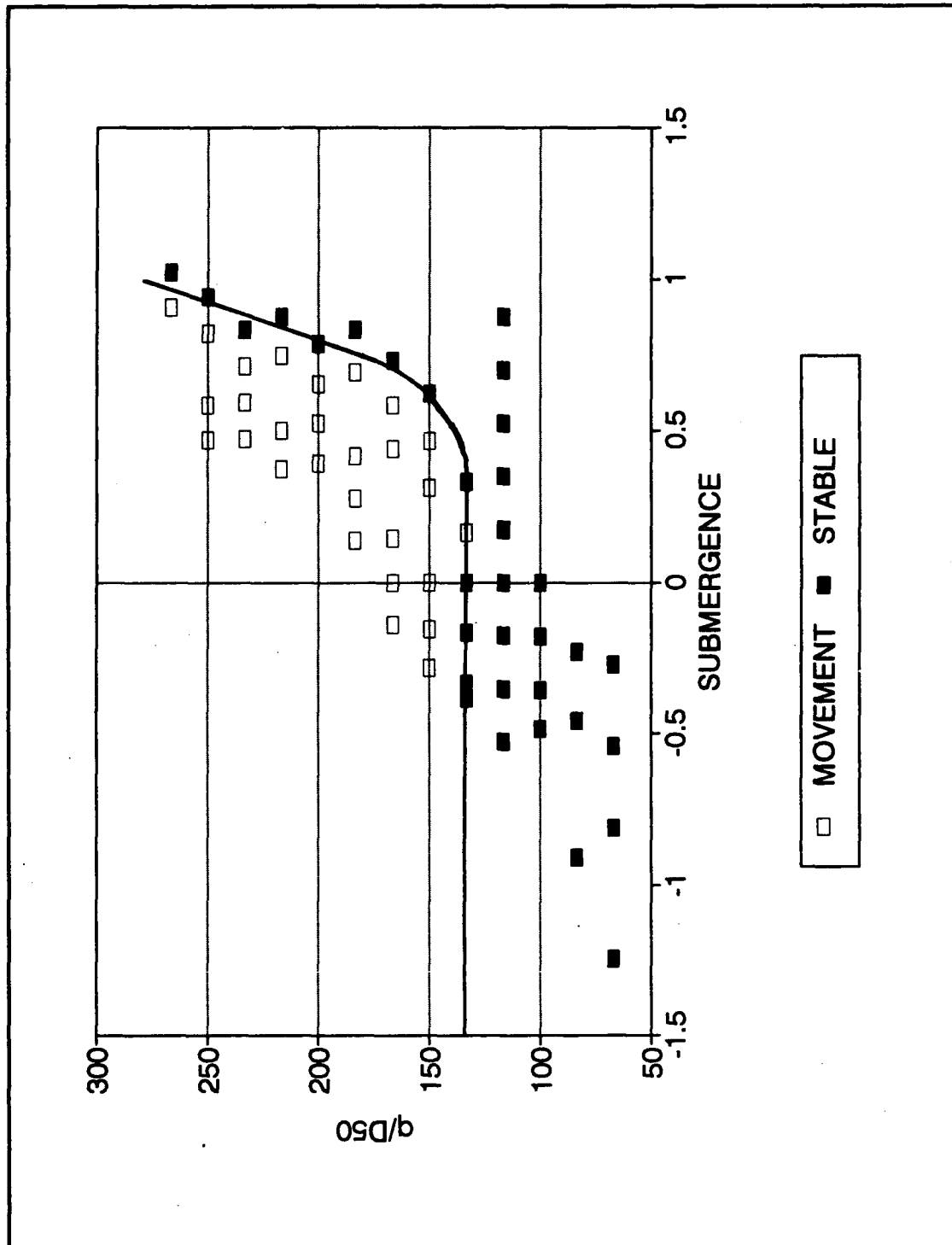
Figure E57. q/d50 versus submergence for small riprap

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Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT (Maximum 200 words)

The purpose of monitoring the Demonstration Erosion Control (DEC) Project is to evaluate and document watershed response to the implemented DEC Project. Documentation of watershed responses to DEC Project features will allow the participating agencies a unique opportunity to determine the effectiveness of existing design guidance for erosion and flood control in small watersheds. The monitoring program includes 11 technical areas: stream gaging, data collection and data management, hydraulic performance of structures, channel response, hydrology, upland watersheds, reservoir sedimentation, environmental aspects, bank stability, design tools, and technology transfer.

A 1:12-scale physical model was used to investigate a proposed sheet-pile grade control structure for the DEC Project in the Yazoo Basin, Mississippi. Low-drop grade control structures have been used as a means to arrest erosion in incising channels. Existing design criteria for these sheet-pile drop structures have been based on a maximum drop of 6 ft. The objective of this study was to determine the feasibility of using a 10-ft drop and develop design guidance pertaining to the higher drop.

A drop structure design based on a study conducted by Colorado State University (CSU) was adopted for the 10-ft drop. An equation developed by CSU for sizing riprap in the stilling basin floor was modified for the higher drop based on results of the physical model test.

14. SUBJECT TERMS
Demonstration Erosion Control (DEC)   Sheet-pile weir
Drop structure   10-ft drop
Riprap

15. NUMBER OF PAGES
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